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# **Observed and Computed Settlements of Structures in Chicago**

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by

Ralph B. Peck

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A REPORT OF AN INVESTIGATION

Conducted by  
THE ENGINEERING EXPERIMENT STATION  
UNIVERSITY OF ILLINOIS

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# **Observed and Computed Settlements of Structures in Chicago**

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### **ABSTRACT**

The settlements of seven structures in downtown Chicago are computed in accordance with the generally recognized procedures of soil mechanics, and the computed settlements are compared with those actually experienced by the structures.

It is found that the order of magnitude of the ultimate settlements and the general pattern of the differential settlements are given with sufficient accuracy for practical purposes by the computations, provided the secondary time effect is excluded from the comparison. The principal deviations between computed and observed settlements are the result partly of unpredictable variations in the compressibility of the subsoil and partly of the restraints to deformation offered by the superstructure.

For a given structure, a simple relationship exists between the average settlement per unit of width and the factor of safety against a bearing-capacity failure.

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## I. INTRODUCTION

### 1. Scope

This bulletin, the third of a series\* dealing with foundation engineering in Chicago, contains the results of a comparison between observed and computed settlements for seven structures located within the central business district. By means of this comparison, conclusions can be drawn concerning the applicability of current methods of settlement computation to soil conditions similar to those in Chicago.

The progressive settlements experienced by the early skyscrapers in Chicago were, in many instances, appreciably greater than the designers anticipated, and were almost universally greater than would be considered tolerable today. After a period of experimentation in the design of spread foundations, the local engineers turned to the use of piles or piers extending through the soft glacial deposits to hardpan or bedrock (Peck 1948). Today, no heavy structures are constructed in the central business district on shallow foundations and, as a consequence, it is seldom necessary that the practicing engineer be able to estimate the settlement caused by compression of the underlying soft clays. Hence, the ability to predict the settlement of structures in this locality on the basis of computations and soil tests is now of little more than academic interest.

Nevertheless, the records of the settlement of the older structures provide invaluable data for investigating the general validity of methods currently in use for making settlement forecasts. For this purpose the records possess an importance far greater than their local origin would seem to indicate.

One of the earliest contributions of soil mechanics to the field of civil engineering was a method for computing the settlement of structures resting above deposits of soft clay. Before the introduction of this method, there were no means for computing or even crudely estimating what the settlement of such structures would be. Estimates based upon judgment and experience with similar buildings often led to disappointing results. This is

borne out not only by the history of foundation engineering in Chicago but also by that in many other localities. Judgment and experience proved unreliable because significant differences in soil properties were not appreciated and because the mechanics of the process of settlement were not yet understood.

The theories and laboratory test procedures required for estimating the settlements of buildings above deposits of soft clay became available in about 1925 (Terzaghi 1925) and were first applied to predict the settlement of a real structure in about 1928. Since that time, many settlement computations have been made. Nevertheless, the reliability of such computations under various conditions has not yet been adequately investigated. This situation is due partly to the fact that the settlement of a building may occur over a long period of time. Often, many years must elapse before sufficient observational data for a new building can accumulate to make possible a comparison between the actual and the computed settlements.

The primary object of this study is to compare the computed and observed settlements of those structures in Chicago for which adequate records are available, in order to permit an evaluation of the degree of reliability of the ordinary methods of settlement computation when applied to conditions similar to those prevailing in that locality. A secondary object is to isolate some of the principal reasons for discrepancies between the computed and observed values.

Settlement forecasts are based on the results of laboratory tests on samples that are completely confined against lateral expansion. Hence, it is reasonable to expect the reliability of settlement computations to be greatest if the subsoil consists of materials that are relatively incompressible except for a limited number of layers of soft clay. Under these conditions, friction and adhesion between the relatively unyielding layers and the intervening soft strata can be expected to prevent most of the lateral deformation of the soft materials. Hence, the conditions of lateral restraint of the soft soil are fairly well defined and it is reasonable to assume that the restraint is complete.

\* University of Illinois Engineering Experiment Station Bulletins 373 and 423.

On the other hand, if the layer of compressible soil is very thick the amount of lateral restraint is problematical. A considerable portion of the settlement may conceivably have its origin in the lateral displacement rather than the consolidation of the soil beneath the structure. The extent to which lateral displacement of the subsoil influences the settlement cannot be ascertained by means of theory or laboratory tests because the conditions of restraint in the field are unknown. The matter can be investigated, however, by comparing the actual settlements of structures with those computed on the assumption that lateral restraint is complete.

Soil conditions in the city of Chicago are such that rather large lateral displacements appear conceivable. The deposit of soft clay is deep; it extends upward to within 2 or 3 ft of footing level; and it is softest near the top. The only factor tending to compensate for the softness of the upper part of the clay is the existence of a relatively stiff upper crust having a thickness of  $2\frac{1}{2}$  to 4 ft.

Fortunately, settlement observations have been made on a number of buildings in Chicago, some for as long as 60 years. In addition, the subsoil was explored extensively during the period 1939-1941 by means of the techniques of modern soil mechanics in connection with the construction of the Chicago subway. The most important data, therefore, are available for the direct comparison of the actual settlements of the structures with those computed on the basis of soil mechanics.

In recognition of this opportunity, the Joint Committee on Soil Mechanics and Foundation Engineering collected all available information concerning soil conditions and settlements of structures. In addition, a search was made to locate foundation plans and other data regarding the design of those structures of greatest interest. Sponsorship of the Joint Committee was a cooperative effort of the Illinois Section of the American Society of Civil Engineers, the Western Society of Engineers, and the Engineering Experiment Station of the University of Illinois. The chairman was Frank A. Randall until his death in 1950, whereupon Verne O. McClurg assumed the chairmanship. The other members were A. E. Cummings, R. B. Peck, F. A. Reickert, P. C. Rutledge, C. P. Siess and K. Terzaghi.

The computations of settlement and much of the work of organizing the data were carried out by

Dr. M. E. Uyanik, whose thesis served as a preliminary draft of this bulletin.

## 2. Computed Settlements

The settlement of a building located above a deposit of soft clay appears to consist of three principal components: an immediate settlement due largely to lateral displacement of the clay beneath the loaded area; a slow inelastic settlement, known as the primary settlement, due to squeezing excess water from the pores of the soil; and a long-time, slow settlement, known as secondary compression, probably caused by a readjustment of the position of the individual grains to the change in stress.

The immediate component of the settlement could be estimated if the elastic properties of the soil in situ could be ascertained. However, it has been demonstrated in several different ways that the modulus of elasticity, for example, of the best undisturbed samples is smaller than that of the undisturbed soil in place. Since no universally reliable means have yet been found for estimating the elastic properties of the natural soil, no method of computation yet developed can be considered reliable. Field evidence indicates that the immediate settlement is usually small compared to the primary settlement that follows.

The procedure for computing the primary settlement involves three separate steps. First, the stresses in the mass of soil due to the weight of the building are estimated. Second, the relation between stress and strain for the soil is established by means of a compression test on a laterally confined sample from which the excess water can drain. Finally, the strains along a vertical line beneath the building are evaluated and integrated to determine the settlement at the ground surface above the vertical line.

The stress in the mass of soil is computed by means of the theory of elasticity, wherein the soil is assumed to be elastic, isotropic, homogeneous, and infinite in extent below a horizontal surface at right angles to which the loads are applied. The vertical pressure at any point within the semi-infinite mass, due to a concentrated load on the surface, can be computed by Boussinesq's equation, and the equation can be integrated to furnish the magnitude of the stress due to a uniformly distributed load. In this study, the stresses have been computed by means of a graphical procedure (Newmark 1942) based on Boussinesq's equation.



The relation between stress and strain for a given soil is determined by means of a consolidation test in the laboratory. A sample, at its natural void ratio,\* is placed in a confining ring and loaded vertically through porous discs that permit the escape of water from the soil during compression. As the vertical pressure is increased, the void ratio decreases from its initial value  $e_0$  to a value  $e$  according to the solid curve shown in Fig. 1. The lower portion of this curve, in a semilogarithmic plot, is fairly straight up to a pressure of 20 to 50 tons per sq ft. It has been observed that the continuation of this straight portion intersects the axis  $e = 0$  at some point  $B$  which, for all practical purposes, has the same position regardless of the degree to which the sample has been disturbed. Hence, it is reasonable to assume that the continuation of the curve representing the stress-strain relation for the perfectly undisturbed soil in the field also passes through this point. Furthermore, it is obvious that at a pressure  $p_0$ , which corresponds to the original stress on the sample when undisturbed in the ground, the void ratio of the untouched clay must have been  $e_0$ . Therefore, the curve representing the relation between  $e$  and  $p$  in the field must pass through point  $A$  with the coordinates  $e_0$  and  $p_0$ . It is assumed that the field  $e$ -log  $p$  relation is a straight line passing between  $A$  and  $B$ . This line is called the virgin compression or virgin consolidation curve. Its slope, as defined in Eq. 1, is designated by  $C_c$ .

The expression for the change in void ratio  $\Delta e$  is

$$\Delta e = C_c \log \frac{p_0 + \Delta p}{p_0} \quad (1)$$

For a given change in void ratio, on the assumption that all the compression takes place in a vertical direction, the settlement  $S$  corresponding to a layer of thickness  $H$  is

$$S = \frac{\Delta e}{1 + e_0} H \quad (2)$$

Since  $\Delta p$  and  $p_0$  vary with depth, and since  $C_c$  may also vary with depth, the clay deposit is divided into layers in each of which the change of stress  $\Delta p$  is practically linear with depth, and in which  $C_c$  may be assumed constant. The values of  $p_0$  and  $\Delta p$  are computed for mid-height of each layer, and the contributions to the total settlement computed by means of Eqs. 1 and 2. The original

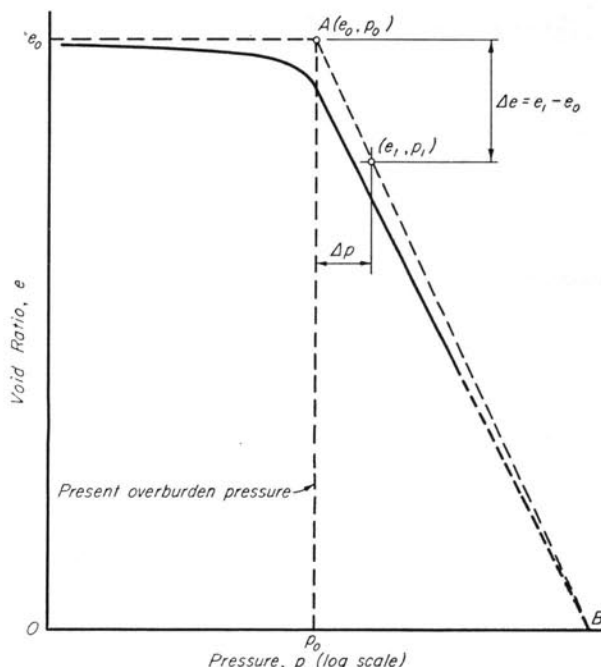


Fig. 1. Typical  $e$ -log  $p$  Curve from Consolidation Test

pressure  $p_0$  is equal to the total weight (soil plus water) of the entire column of soil above the given depth, minus the weight of the column of water extending from groundwater level to the given depth. The added stress  $\Delta p$  is equal to that produced by the footing loads (positive), by any back-fill above footing level (positive), and by the weight of soil removed by excavation (negative).

The soil constant  $C_c$  can be directly determined only by consolidation tests. However, consolidation tests had not been performed on samples in sufficient number from the site of each building considered in this study to provide representative values of  $C_c$  at each site. Fortunately, the values of  $C_c$  for clays in the downtown district of Chicago are known to be closely related to those of the natural water content or initial void ratio.\* A linear relation exists between the quantities  $e_0$  and the compression ratio  $C_c/(1 + e_0)$ , as shown in Fig. 2 (Peck and Reed 1954). By means of this relation,  $C_c$  can be found from the nearest subway borings by the following procedure: from the borings, which include the results of water-content determinations, an average value of  $w$  for a given layer is determined. The value of  $e_0$  is computed by means of the expression  $e_0 = 2.8w$ , whereupon  $C_c$  is determined from Fig. 2.

\* The void ratio  $e$  is defined as the volume of voids divided by the volume of solid matter.

\* The water content  $w$  is equal to the weight of water divided by the dry weight of the soil mass. For a saturated soil, the water content and void ratio are related by the expression  $e = gw$ , wherein  $g$  is the specific gravity of the solid soil constituents.

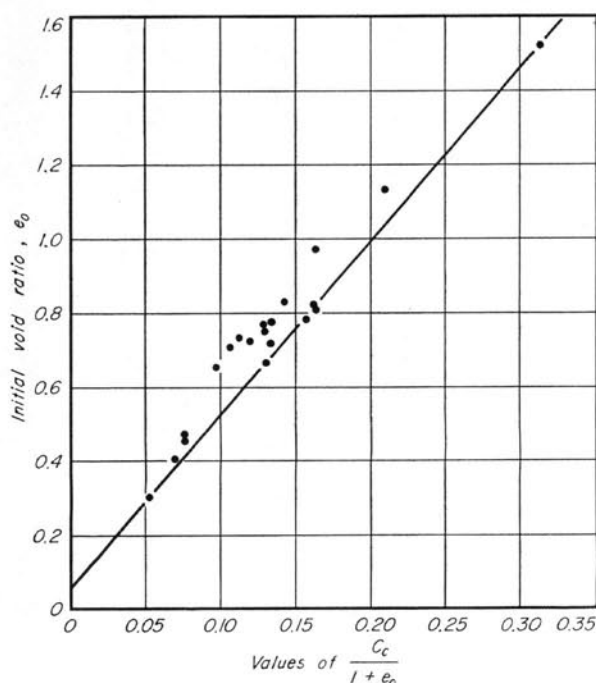


Fig. 2. Relation Between Void Ratio and Compression Ratio

The third constituent of the settlement, the so-called secondary compression or secondary time effect, cannot yet be computed because the factors upon which it depends are not yet known. As long as an appreciable part of the added stress is carried by the porewater in the soil, the time-settlement relation for a consolidating clay is governed by the laws of hydraulics, and these laws form the basis of the theory of consolidation (Terzaghi 1943). According to this theory, the time-settlement curves should approach asymptotically some final value of settlement. However, even in laboratory tests, the time-settlement curves approach inclined tangents and settlement continues after the porewater pressure becomes negligible. In order to construct the  $e$ -log  $p$  curve shown in Fig. 1, some procedure must be adopted to exclude from the total change in void ratio that part which is due to the additional or secondary settlement, because the magnitudes of the secondary settlement in the field and laboratory appear to be unrelated. A graphical procedure is commonly used (Casagrande and Fadum 1940) to accomplish this end. It is based on a comparison of many laboratory time-settlement curves with the curve derived from the theory of consolidation. By means of Casagrande's procedure, the settlement corresponding to the end of primary consolidation (often called "theoretical 100 percent consolidation") is determined.

The settlement corresponding to 100 percent consolidation is determined by tracing the curve representing the relation between settlement and the logarithm of the time for any given increment of pressure. The characteristic shape of this curve is shown in Fig. 3. Two tangents are drawn to the curve. One is the upward extension of the straight lower portion; the other is drawn at the point of inflection. The ordinate of their point of intersection is presumed to correspond to 100 percent consolidation.

The abscissa of the point determining the settlement at 100 percent consolidation has no physical meaning, because theoretically the time required to reach 100 percent consolidation is infinite. However, in a loose way, the time corresponding to this abscissa is a measure of the time required for virtual completion of the primary consolidation. It will be so used in this study for interpreting the actual settlement curves of some of the structures.

In this study, the term "computed settlement" refers only to the primary settlement of a laterally confined soil. Because of the procedures used in the interpretation of routine consolidation tests, the initial settlement due to lateral displacement is excluded. By means of the graphical construction outlined in the preceding paragraph, the secondary settlement is also excluded. These exclusions, necessary because of the present limited state of knowledge of the behavior of soils, place certain limitations on the reliability of settlement forecasts. They also restrict the useful field evidence for purposes of the present study to settlement data in which at least the secondary compression can be segregated from the primary.

### 3. Observed or Actual Settlements

It is evident from the preceding discussion that, since the settlement of a building may continue for a long time, some point on the time-settlement curve for a given reference point must be selected for comparison with the computed settlement. Furthermore, if only one set of level readings is available, taken at a given time after construction of the building, some basis must be found for ascertaining at least roughly how these settlements would compare with those corresponding to the time when primary consolidation was almost complete.

In the studies to be described, the graphical construction of Casagrande was used to permit elimination of the secondary settlement whenever the time-settlement curve for at least one point of an

actual building was available. Since Casagrande's procedure was developed for laboratory conditions, its application to field data is arbitrary. However, its use is reasonable and puts the consideration of all buildings on the same basis.

The magnitude of the settlement remaining after elimination of the secondary time effect is referred to in this bulletin as the "observed" or "actual" settlement, in contrast to the "total observed settlement" which is taken to mean the real settlement of the building at the time a given set of levels was taken.

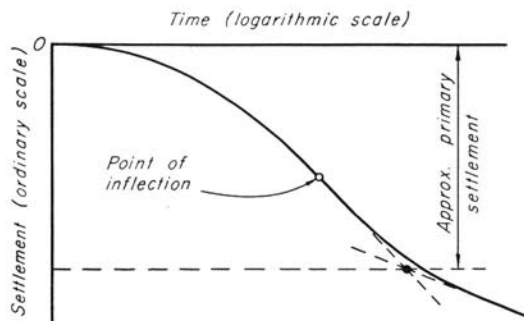


Fig. 3. Typical Semi-Logarithmic Time-Settlement Curve from Consolidation Test on Clay

The time-settlement curve for one point in a building does not necessarily have the same characteristics as that of another, and the ratio of the observed to the total observed settlement may not be a constant. However, the ratio usually does not vary greatly for the points within a given building, and if only one time-settlement curve was available, the same ratio was assumed to apply to all points included in a settlement survey of a given date.

In some instances, no time-settlement data for a building were available, but settlement contours could be drawn for a certain date a given number of years after construction. In these instances it was necessary to estimate the degree to which consolidation had progressed at the time of observation. It should be possible to compute, by means of the theory of consolidation, the time required to reach any given fraction of the primary settlement, provided the initial distribution of consolidation stress, the drainage conditions, the thickness of the consolidating layer, and the coefficient of consolidation are known. The last factor is fairly constant for Chicago clays; furthermore, the drainage conditions of the Chicago subsoil are relatively uniform and the thickness of the compressible layers fairly con-

stant. Hence, it seemed likely that some simple expression, based on theory, could be derived to relate the times required by various buildings to reach a given degree of consolidation. However, no such relation could be found. Indeed, in many instances the shape of the time-settlement curves hardly resembles the theoretical curves. Therefore, only a rough estimate of the ratio of observed to total observed settlements can be given for those buildings lacking time-settlement data.

In summary, the observed settlement includes those components due to lateral deformation and to primary (purely vertical) consolidation; it excludes the secondary compression. The computed settlement includes only the purely vertical, primary consolidation. Hence, any discrepancies between the two are at least to some degree a measure of the importance of the settlements associated with imperfect lateral restraint.

#### 4. Buildings Considered

Settlement computations were made for seven buildings. They are discussed in the following order:

Chicago Auditorium  
Masonic Temple  
Monadnock Block  
Old Board of Trade  
Apartment Building  
Polk Street Station  
Judson Warehouse

All these buildings are located near or within the Loop area, as shown in Fig. 4. The dates of construction range from 1884 to 1929. The magnitudes of the maximum total settlement range from 36 to 31¼ in.

The foundations consist of individual spread footings of various sizes, or of mats under part or all of a building. All of the buildings have full basements except the Judson Warehouse, which has a basement under only one end. More specific information will be given for each building when it is discussed individually.

#### 5. General Soil Conditions under Loop Area

The ground surface in the Loop area is now at approximately El. +14.00 C.C.D. (Chicago City Datum, equal to El. 579.94 mean sea level, and roughly equal to the level of Lake Michigan). Before 1856, the elevation of the Loop was about +6 C.C.D. From time to time, the ground surface was raised until in the 1880's it reached its present eleva-



Fig. 4. Location of Structures Included in Study

tion. Hence, the top eight feet of soil consist of various fills. The groundwater table is roughly at El. +4.\* The remaining four feet, down to city datum, consist usually of sand and sandy silt. Below datum is a crust of tough clay with a thickness varying from about 2.5 to 4.0 ft. Under this crust is found a deep plastic glacial clay deposit with a thickness from 35 to 45 ft. It varies in consistency from very soft to stiff. Under this deposit is a bed of hard clay that may extend to bedrock or that may be underlain by gravel and sand. The surface of the rock is found at elevations varying from about -70 to -120. A more complete description of the subsurface conditions is given in Peck and Reed (1954).

#### 6. General Assumptions for All Buildings

Since all the clay layers, which are the main source of settlement, lie below the groundwater table (El. +4) all the soil below this elevation

\* All elevations given from now on are referred to Chicago City Datum.

was assumed to be fully saturated. The initial void ratio of every layer was determined by means of the equation  $e_0 = 2.8w$ , where  $w$  is the water content in percent of dry weight and the factor 2.8 represents the specific gravity of the solid soil particles.

The saturated unit weight of the clay was determined by the formula

$$\gamma = \frac{174.7(1 + w)}{1 + 2.8w} \quad (3)$$

where  $\gamma$  is the unit weight of the soil in lb per cu ft.

The unit weight of all fill materials was assumed to be 110 lb per cu ft. However, the saturated soil between elevations +4 and 0 was assumed to weigh 126.0 lb per cu ft. In every instance the clay was assumed to be fully consolidated under its effective overburden pressure before the weight of the building was added.

In the computation of stresses, the surface of the semi-infinite mass was assumed to be at the level of the base of the footings. Furthermore, each building was assumed to be perfectly flexible.

For values of final pressure less than the initial value  $p_0$ , the value of  $C_c$  was taken equal to zero. That is, the clay was assumed not to swell.

Any additional assumptions made for computing settlements are stated when the buildings are discussed individually.

It will be noted that the assumptions and procedures for the computation of settlement are uniform and consistent for all seven structures. For each structure, somewhat better agreement between computed and observed settlements might have resulted if one or more of the assumptions or procedures had been modified in an entirely justifiable manner. However, since the designer of a new structure does not have the benefit of the hindsight provided by settlement observations on his structure, it was felt that a fairer and more realistic comparison would be obtained if no advantage were taken of knowledge of special conditions that would have remained unknown if the history of the building had not been preserved.



## II. CHICAGO AUDITORIUM

### 7. Information Available

The Chicago Auditorium, Fig. 5, is located between Michigan Blvd and Wabash Ave on the north side of Congress St. It is roughly rectangular in shape, extending 160 ft along Wabash Ave and Michigan Blvd and 360 ft along Congress St. The portion fronting on Wabash Ave and part of that on Congress St form the office building. The Michigan Blvd front and the remainder of that on Congress St comprise the hotel. The entire area enclosed by the portions facing these three streets is occupied by the theater, which has its stage toward Michigan Blvd and its balcony toward Wabash Ave. The theater entrance is on the Congress St side about one quarter of the distance from Wabash Ave to Michigan Blvd. At this location rises a 19-story tower. The remainder of the structure is 10 stories.

The building is constructed with masonry bearing walls, which are continuous along the back of the building parallel to Congress St, and for the most part in the interior. The Michigan Blvd, Congress St and Wabash Ave fronts, however, consist of separate piers and columns in the lower stories. Interior loads in the hotel and office building and in the tower are carried by cast iron columns and steel or iron floor beams. Columns are also located in the foyer at the rear of the theater. Within the theater itself there are no columns except those used to support the floor. The roof is supported by wrought iron trusses resting upon masonry walls.

The general excavation level was approximately 12 ft below the surface. The footings themselves were 17 ft deep (El. -3.0) with the exception of that for the tower. Excavation for the tower extended 18 ft below the surface. All footings consisted of two layers of 12-in. timbers resting on a prepared base of gravel. The second layer of timbers was laid at right angles to the first. Above the timbers was a stepped grillage of concrete interlaced with railroad rails. Resting on the concrete were rubble masonry walls extending to the street grade. Usually, a separate footing was used under each wall and column. The foundation for the tower was one large footing approximately 100 by

67 ft. It also consisted of a timber grillage and concrete mat, and of individual piers resting upon the mat to support the walls and columns. In order to connect the tower to the adjacent footings on the front wall, a series of 15-in. steel beams extended from the lower part of the tower footing into the adjacent footings. The basement floor was established on a fill at approximately El. +6.5.

No footing plan for the Auditorium has been located. However, fragmentary information has been obtained from construction photographs and diagrams of footing details in an article by one of the architects, Dankmar Adler, in the *Inland Architect* of March 1888. Further data can be found in the records of court testimony\* of several engineers who had been employed in the design or construction of the building. On the basis of all the evidence, the footing plan shown in Fig. 6 has been drawn and is believed to represent the actual conditions quite accurately. Mr. Adler stated that a design load of 4500 lb per sq ft was adopted, of which it is likely that about 4100 lb per sq ft constitute dead load.

Construction of the building started in August 1887, and was completed in February 1890. The building was set 2½ in. above grade to allow for settlement.

Settlement reference points were set at the numbered positions shown in Fig. 7 by Mr. M. L. Greeley in August 1887 on the tops of the stone caps of the piers before the piers received any additional load. The elevations of these points have been observed at various intervals by Mr. Greeley and his associates until the present time. The results of these observations are shown in Table 1. In 1941 a series of differential levels was run inside the foyer by the Joint Committee and correlated with the exterior levels taken by Mr. Greeley. The results of all the observations made in 1941 are indicated graphically on Fig. 7. Before 1909, the bench mark was located on a building on the opposite side of Wabash Ave and may have settled. It

\* Official Transcript, District Court of the United States, Northern District of Illinois, Eastern Division, Dec. 7, 1925.



Fig. 5. The Chicago Auditorium

is known that its settlement was less than 1 in. After 1909, levels were referred to a bench mark on the McCormick Building, founded on piers to bedrock.

The site of the Auditorium had been previously occupied only by small buildings not exceeding two stories. The Fine Arts building, an 8-story structure adjacent to the Auditorium on the north, had been completed and was underpinned during construction of the Auditorium. No structures of importance were located across any of the streets from the building, and since construction of the building no excavation for deep foundations has been made in the vicinity. The nearest large building is the Congress Hotel, a 10-story structure on spread foot-

ings, built in 1893. It is 80 ft distant from the Auditorium and has a one-story basement.

The stage of the Auditorium theater is supported on 28 hydraulic jacks located in wells 36 ft deep excavated in the clay according to plans proposed by General Wm. Sooy Smith. During the excavation of these wells, dug after the foundations were complete, a settlement of about 4 in. occurred in adjacent walls.

A subway boring was made in 1939 on the opposite side of Wabash Ave from the northwest corner of the Auditorium.

#### 8. Assumptions for Computation

Although there is a difference of 1.0 ft in the elevations of the base of the tower footing and of

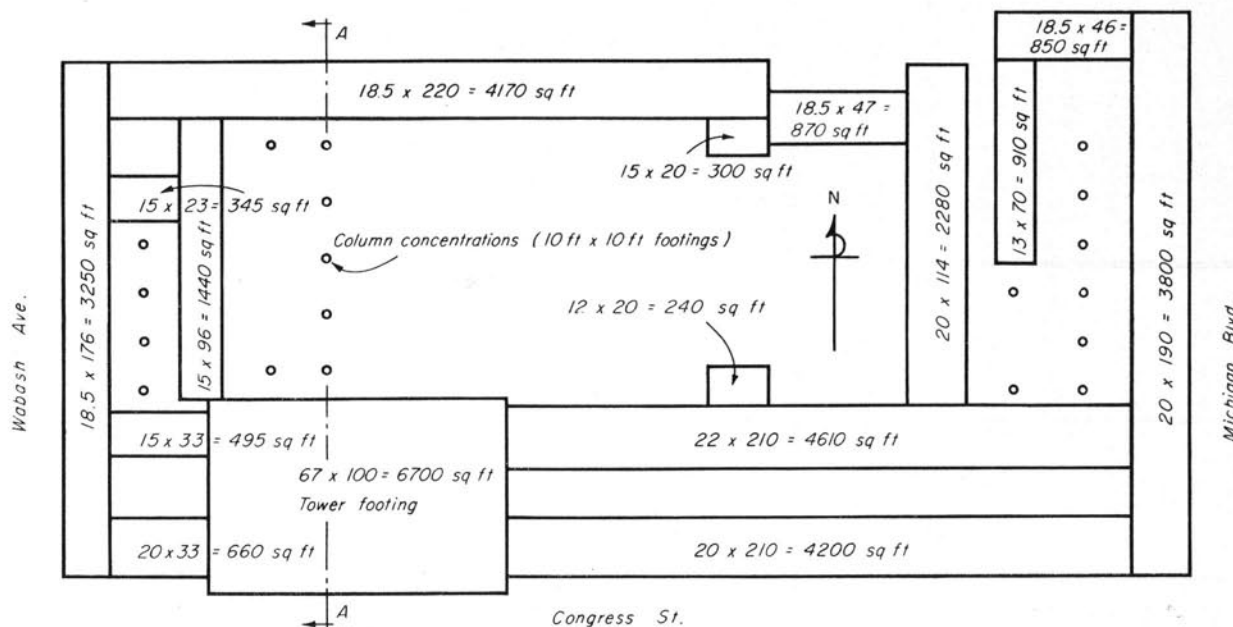


Fig. 6. Reconstructed Foundation Plan of Auditorium

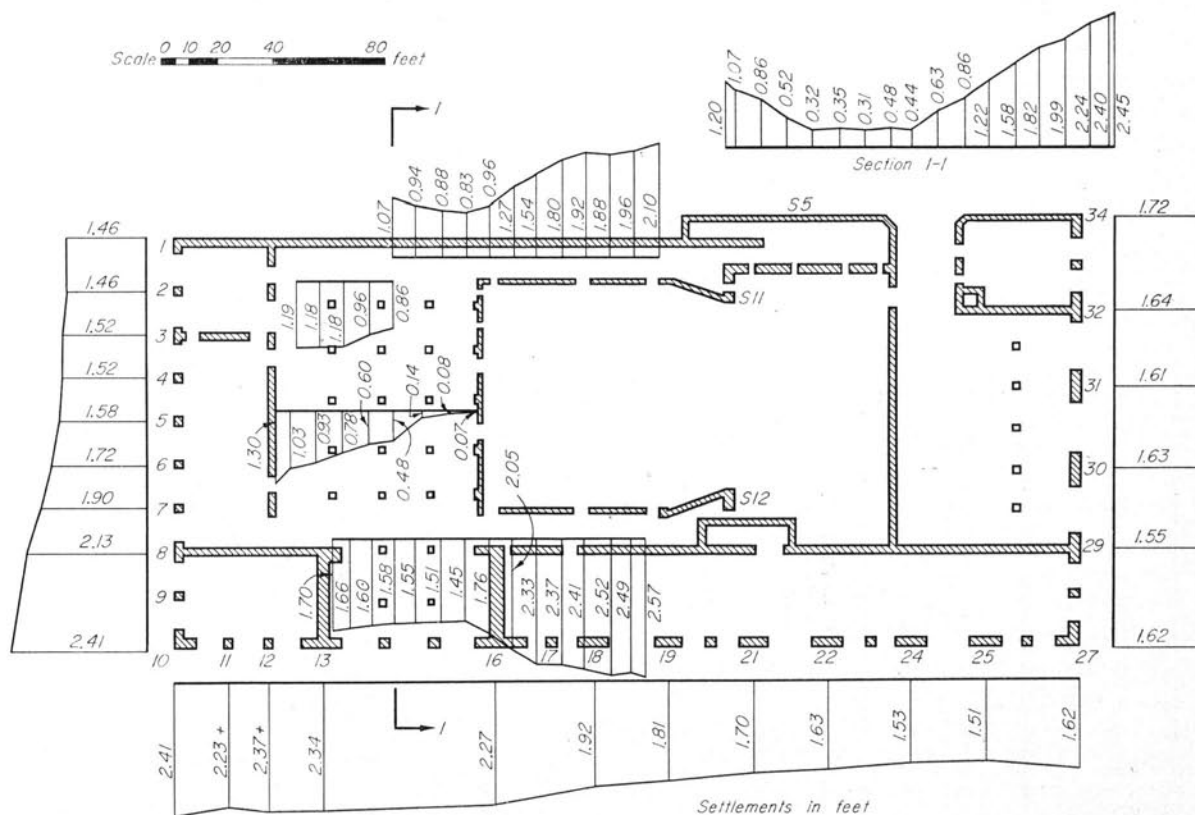


Fig. 7. Settlements of Auditorium in 1941

the individual column footings, El. -3.0 was assumed to be the base of all the footings. This assumption was made to simplify the computation of stresses under the building. It has a slight effect on the stresses in the uppermost layers, but in view

of the other assumptions concerning the foundation plan this error in the stresses is negligible.

The log of the subway boring is shown in Fig. 8. The water contents of the layers below El. -46 were estimated as 14.5 percent for stiff clay (see

Table 1  
Measured Settlements of Auditorium  
Settlements in feet

Point	Orig. Elev.	8-87	8-88	1-89	7-89	1-90	6-90	1-91	6-91	1-92	7-92	1-93	8-93	3-94	1-95	7-95	3-96	7-96	1-97	7-97
1	15.40	0	.25	.41	.51	.60	.67	.72	.75	.78	.84	.88	.89	.91	.94	.97	1.00	1.01	1.04	1.04
3	15.37	0	.15	.31	.44	.54	.64	.69	.72	.79	.82	.87	.91	.92	.98	.98	1.03	1.05	1.05	1.08
4	15.37	0	.11	.27	.41	.52	.61	.68	.71	.76	.81	.86	.89	.92	.96	.99	1.02	1.04	1.04	1.06
5	15.36	0	.11	.30	.46	.57	.67	.73	.78	.83	.87	.93	.97	.99	1.04	1.08	1.10	1.12	1.13	1.16
6	15.37	0	.13	.33	.48	.61	.69	.77	.83	.90	.94	1.00	1.03	1.06	1.11	1.14	1.17	1.19	1.19	1.21
7	15.38	0	.13	.36	.54	.69	.81	.86	.96	1.01	1.08	1.16	1.18	1.21	1.27	1.31	1.33	1.35	1.37	1.39
8	15.38	0	.17	.42	.63	.78	.93	1.01	1.10	1.19	1.25	1.33	1.37	1.39	1.48	1.50	1.55	1.57	1.59	1.60
10	15.38	0	.12	.45	.73	.97	1.06	1.16	1.27	1.32	1.43	1.49	1.57	1.59	1.67	1.72	1.73	1.77	1.80	1.82
11	15.42	0	.10	.41	.66	.83	.98	1.19	1.20	1.26	1.34	1.38	1.48	1.52	1.57	1.63	1.67	1.68	1.70	1.75
12	15.39	0	.16	.40	.69	.85	1.12	1.23	1.33	1.39	1.47	1.50	1.61	1.65	1.70	1.76	1.78	1.82	1.83	1.88
13	15.37	0	.12	.43	.68	.85	1.03	1.09	1.25	1.30	1.41	1.43	1.52	1.57	1.63	1.70	1.73	1.73	1.76	1.80
16	15.45	0	.16	.43	.68	.90	1.04	1.16	1.25	1.34	1.34	1.41	1.52	1.55	1.59	1.67	1.69	1.69	1.71	1.73
18	15.38	0	.06	.32	.57	.79	.87	1.01	1.04	1.07	1.14	1.19	1.23	1.24	1.28	1.32	1.38	1.38	1.38	1.41
19	15.39	0	.05	.31	.52	.75	.77	.86	.95	.98	1.03	1.08	1.10	1.12	1.16	1.21	1.25	1.25	1.25	1.28
21	15.39	0	.05	.25	.45	.65	.77	.87	.89	.92	.98	1.08	1.08	1.09	1.11	1.16	1.19	1.19	1.19	1.23
22	15.39	0	.05	.23	.43	.63	.77	.87	.89	.92	.98	1.08	1.08	1.09	1.11	1.16	1.19	1.19	1.19	1.23
24	15.38	0	.06	.19	.42	.58	.70	.80	.85	.87	.93	.96	.98	1.02	1.03	1.08	1.12	1.12	1.12	1.16
25	15.39	0	.05	.20	.40	.58	.68	.77	.83	.89	.91	.95	.95	.99	1.02	1.06	1.10	1.09	1.11	1.14
27	15.39	0	.09	.29	.49	.68	.75	.85	.91	1.05	1.07	1.07	1.08	1.10	1.14	1.19	1.20	1.24	1.24	1.26
29	15.40	0	.05	.27	.50	.69	.77	.86	.92	1.00	.98	1.03	1.04	1.07	1.11	1.16	1.15	1.17	1.20	1.20
31	15.40	0	.05	.33	.56	.72	.82	.89	.91	.95	1.01	1.05	1.07	1.08	1.11	1.13	1.19	1.19	1.18	1.22
32	15.41	0	.04	.36	.56	.70	.84	.87	.92	.96	.96	1.02	1.16	1.08	1.10	1.14	1.19	1.19	1.18	1.22
34	15.38	0	.09	.32	.52	.69	.77	.85	.89	.93	.94	.97	1.03	1.08	1.10	1.14	1.19	1.19	1.18	1.22
S5	21.66	0	.16	.66	.70	.70	.70	.70	.70	.70	.70	.70	.70	.70	.70	.70	.70	.70	.70	.70
S11	12.26	0	.17	.30	.58	.79	.93	.92	.98	.97	1.03	1.03	1.05	1.14	1.14	1.17	1.18	1.22	1.28	1.25
S12	12.20	0	.20	.38	.70	.85	1.06	1.17	1.15	1.19	1.31	1.31	1.31	1.35	1.38	1.43	1.46	1.46	1.47	1.52

Fig. 8) and 12.0 percent for hard clay. The entire clay deposit under the building (from El. -3.0 to -71.0) was divided into seven layers, A to G inclusive, Fig. 9.

Since this was the first building to be studied in the seven-building series, a preliminary investigation was made to determine which layers of the subsoil were responsible for the greater part of the settlement. For this purpose, the entire surface of the ground over an infinite area was assumed to be loaded successively at intensities of 0.5, 1.0, 2.0 and

4.0 tons per sq. ft. Such a loading produces constant stresses of the same magnitude as the surcharge in all the layers. After the settlements were computed, the percentage of settlement contributed by each layer was found (see Table 2). The results show that 80 percent of the total settlement is contributed by layers B, C, D and E (Fig. 9)

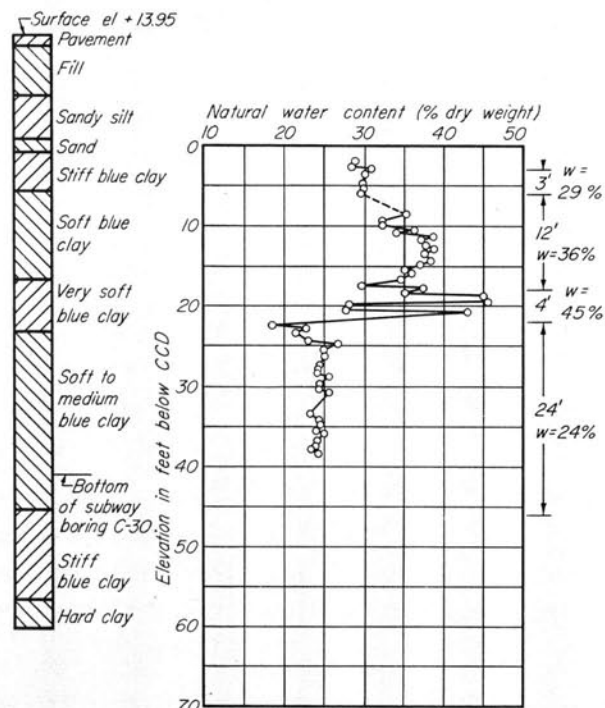


Fig. 8. Log of Boring near Auditorium

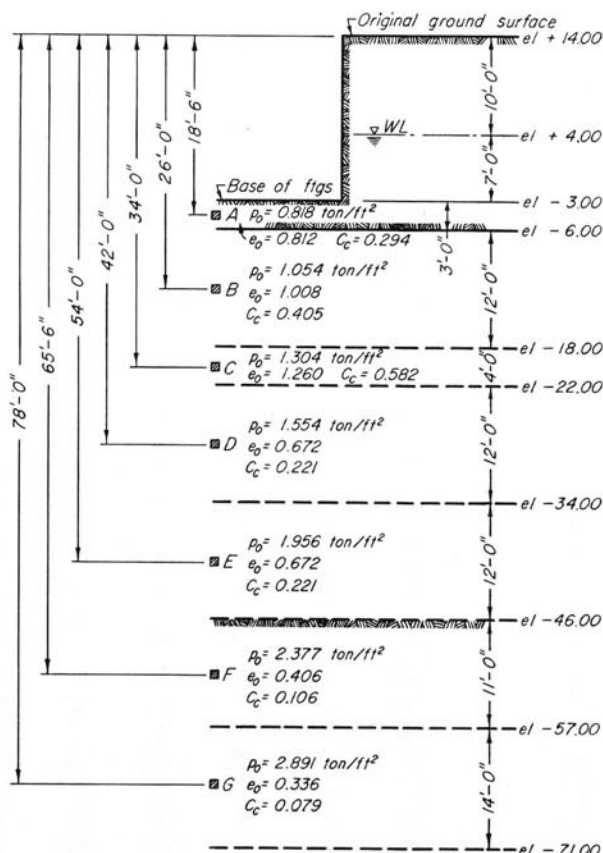


Fig. 9. Data for Computation of Settlement of Auditorium



Table 1 (Continued)  
Measured Settlements of Auditorium  
Settlements in feet

5-00	7-01	8-02	8-04	8-07	11-08	8-12	4-15	4-17	4-18	5-19	5-21	9-23	10-24	10-25	12-26	8-27	2-32	4-33	8-41
1.15	1.16	1.18	1.19	1.23	1.21	1.24	1.24	1.29	1.29	1.32	1.34	1.37	1.37	1.43	1.45	1.45	1.45	1.46	1.46
1.17	1.20	1.22	1.23	1.25	1.25	1.28	1.29	1.33	1.33	1.35	1.39	1.43	1.43	1.46	1.47	1.47	1.47	1.50	1.50
1.15	1.18	1.21	1.23	1.30	1.26	1.28	1.30	1.36	1.36	1.39	1.40	1.45	1.46	1.46	1.49	1.51	1.50	1.50	1.52
1.22	1.24	1.27	1.29	1.33	1.33	1.36	1.38	1.43	1.43	1.45	1.45	1.50	1.51	1.52	1.54	1.55	1.55	1.55	1.58
1.33	1.36	1.40	1.41	1.49		1.48	1.53	1.55	1.55	1.58	1.60	1.66	1.66	1.68	1.70	1.70	1.71	1.71	1.72
1.46	1.48	1.50	1.54	1.60	1.61	1.64	1.64	1.66	1.69	1.72	1.74	1.84	1.84	1.86	1.86	1.88	1.88	1.88	1.90
1.73	1.75	1.77	1.80	1.87	1.89	1.91	1.92	1.93	1.95	1.97	2.00	2.09	2.10	2.08	2.09	2.10	2.10	2.10	2.13
1.94	1.96	1.99	2.06	2.14	2.15	2.17		2.23					2.32	2.38	2.39	2.39	2.40	2.41	
1.84	1.89	1.91	1.95	2.03	2.05	2.07	2.09	2.12	2.14	2.16	2.20	2.23	2.23						
1.96	2.02	2.05	2.08	2.16	2.17	2.20	2.24	2.26	2.27	2.30	2.33	2.37	2.37						
1.89	1.94	1.96	2.00	2.05	2.07	2.12	2.16	2.18	2.18	2.21	2.23	2.28	2.30	2.31	2.31	2.31	2.33	2.33	2.34
1.82	1.85	1.88	1.93	1.97	2.01	2.07	2.13	2.21	2.13	2.14	2.16	2.22	2.23	2.25	2.27	2.27	2.27		
	1.59	1.63	1.68	1.72	1.74	1.74	1.76	1.78	1.79	1.82	1.84	1.88	1.89	1.90	1.90	1.91	1.91	1.91	1.92
1.48	1.52	1.53	1.58	1.63	1.63	1.63	1.63	1.66	1.67	1.70	1.71	1.76	1.78	1.78	1.79	1.79	1.80	1.81	1.81
	1.35	1.42	1.47	1.49	1.52	1.52	1.52	1.55	1.55	1.58	1.60	1.64	1.65	1.67	1.67	1.68	1.70	1.70	1.70
	1.32	1.34	1.35	1.39	1.44	1.44	1.44	1.47	1.48	1.50	1.53	1.56	1.58	1.59	1.59	1.61	1.62	1.62	1.63
	1.24	1.24	1.28	1.29	1.33	1.33	1.33	1.36	1.37	1.39	1.42	1.46	1.47	1.47	1.47	1.48	1.50	1.50	1.53
	1.20	1.22	1.26	1.29	1.32	1.32	1.32	1.34	1.35	1.36	1.40	1.44	1.45	1.46	1.46	1.47	1.48	1.48	1.51
	1.34	1.37	1.36	1.38	1.41	1.41	1.41	1.45	1.45	1.48	1.50	1.54	1.55	1.56	1.56		1.56	1.60	1.62
	1.31	1.31	1.34	1.37	1.39	1.39	1.39	1.44	1.44	1.47	1.50	1.51	1.51	1.51	1.54	1.55			
	1.33	1.36	1.37	1.40	1.43	1.43	1.43	1.48	1.48	1.50	1.52	1.52	1.55	1.57	1.57	1.60	1.61	1.61	1.61
	1.34	1.37	1.38	1.41	1.44	1.44	1.44	1.50	1.50	1.51	1.53	1.55	1.56	1.58	1.58	1.63	1.63	1.64	1.64
	1.36	1.40	1.44	1.47	1.52	1.53	1.53	1.57	1.58	1.61	1.65	1.66	1.68	1.68	1.68	1.70	1.71	1.72	1.72
1.31	1.33	1.39	1.40	1.44	1.48	1.48	1.48	1.55	1.56	1.55	1.58	1.61		1.61	1.61	1.66			
1.35	1.37	1.39	1.44	1.55	1.49	1.53	1.53	1.56	1.56	1.59	1.60	1.65		1.67	1.68	1.68	1.69	1.69	1.74
1.58	1.60	1.61	1.69	1.65	1.69	1.72	1.73	1.74	1.74	1.77									

between El. -6.0 and El. -46.0. The two bottom layers contribute not more than 7.0 percent each at any intensity of loading. Furthermore, since the surcharge of infinite extent produces far greater stresses in these two layers than would be produced if the loads extended merely over the finite area of an actual building, the contribution of these layers to the total settlement would be considerably less than 7.0 percent under an actual building which covers a rather limited area. Finally, the stiff clay

crust, layer A (Fig. 9), between El. -3.0 and El. -6.0, is actually strongly overconsolidated and much less compressible than assumed on the basis of its water content. It undoubtedly contributes less than the calculated 10 percent of the total settlement. Hence, in subsequent computations, these three layers were assumed to make no contribution to the total settlement; i.e., they were assumed to be incompressible.

These assumptions slightly reduced the computed settlements. However, they appreciably decreased the labor in making stress computations for the various structures.

Stresses beneath the Auditorium were computed under 18 points along the line A-A, Fig. 6, at nine different elevations (See Table 3). The soil pressure was taken as 4100 lb per sq ft. A curve was drawn to represent the variation of pressure with depth beneath each point and values of  $\Delta p$  at the center of each layer were scaled. The settlement contributed by each layer was then calculated. The sum of all these settlements for each layer was equal to the computed primary settlement at the given point. Figure 10 shows the profile of computed settlements along line A-A (Fig. 6), together with pressure bulbs under each footing for values of pressure corresponding to the unconfined compressive strength  $q_u$  of each layer. This diagram facilitates a comparison of the stresses with the strength of the clay. The strengths shown are those of 2-in. thin-walled tube samples which were slightly disturbed. The corresponding values for undisturbed samples would be approximately 1.35 times greater (Peck and Reed 1954).

Table 2  
Settlements of Each Layer beneath Auditorium  
due to Surcharges of Infinite Extent

$\Delta p$	Layer	$\Delta e$	$e_0$	$\frac{\Delta e}{1+e_0}$	H	$\Delta S$	%S
0.50	A	0.065	0.812	0.0358	3	0.1074	8.68
	B	0.076	1.008	0.0378	12	0.4536	36.66
	C	0.082	1.260	0.0362	4	0.1448	11.70
	D	0.030	0.672	0.0179	12	0.2148	17.39
	E	0.024	0.672	0.0144	12	0.1728	13.99
	F	0.009	0.406	0.0064	11	0.0704	5.69
	G	0.007	0.336	0.0052	14	0.0728	5.89
					Total	1.2366	100.00
1.0	A	0.105	0.812	0.0580	3	0.1740	8.33
	B	0.124	1.008	0.0617	12	0.7404	35.37
	C	0.146	1.260	0.0646	4	0.2584	12.34
	D	0.051	0.672	0.0305	12	0.3660	17.50
	E	0.043	0.672	0.0257	12	0.3084	14.73
	F	0.014	0.406	0.0100	11	0.1100	5.25
	G	0.013	0.336	0.0097	14	0.1358	6.48
					Total	2.0930	100.00
2.0	A	0.160	0.812	0.0883	3	0.2649	7.91
	B	0.193	1.008	0.1382	12	1.1520	34.38
	C	0.234	1.260	0.1080	4	0.4320	12.90
	D	0.082	0.672	0.0480	12	0.5760	17.20
	E	0.070	0.672	0.0418	12	0.5016	14.98
	F	0.026	0.406	0.0185	11	0.2035	6.07
	G	0.021	0.336	0.0157	14	0.2198	6.56
					Total	3.3498	100.00
4.0	A	0.229	0.812	0.1264	3	0.3792	7.52
	B	0.278	1.008	0.1382	12	1.6584	32.95
	C	0.353	1.260	0.1563	4	0.6252	12.41
	D	0.123	0.672	0.0736	12	0.8832	17.53
	E	0.110	0.672	0.0659	12	0.7908	15.71
	F	0.044	0.406	0.0313	11	0.3443	6.83
	G	0.034	0.336	0.0254	14	0.3556	7.05
					Total	5.0367	100.00

$\Delta p$  = Net pressure added at footing level (tons/ft<sup>2</sup>)

$e_0$  = Initial void ratio

$\Delta e$  = Change in void ratio due to pressure  $\Delta p$

H = Thickness of layer (ft). For location of layer, see Fig. 9.

$\Delta S$  = Increment of settlement due to compression of layer (ft)

%S = Percentage contributed by layer to total settlement

Table 3  
Net Vertical Pressures beneath Auditorium and Computed Settlements

Point	Pressure (ton/ft <sup>2</sup> ) at depth indicated (ft)										Settlement (ft)
	0	3	6	9	12	15	18	22	34	46	
a	1.150	1.150	1.150	1.145	1.105	1.050	1.038	1.015	0.850	0.730	1.593
b	1.150	1.150	1.150	1.150	1.130	1.080	1.060	1.045	0.882	0.730	1.625
c	1.150	1.150	1.150	1.150	1.150	1.140	1.125	1.125	0.952	0.785	1.679
d	1.150	1.150	1.150	1.150	1.150	1.150	1.145	1.120	1.012	0.857	1.705
e	1.150	1.150	1.150	1.150	1.150	1.132	1.105	1.065	0.921	0.769	1.642
f	1.150	1.150	1.150	1.115	1.085	1.028	0.972	0.907	0.780	0.664	1.530
g	1.150	0.880	0.795	0.717	0.698	0.662	0.632	0.583	0.555	0.505	1.121
h	1.150	0.660	0.404	0.290	0.218	0.211	0.206	0.201	0.321	0.336	0.454
j	-0.150	-0.388	-0.226	-0.116	-0.099	-0.067	-0.076	-0.031	0.055	0.214	0.003
k	1.150	0.673	0.370	0.186	0.010	-0.056	-0.092	-0.139	-0.041	0.093	0.183
l	-0.400	-0.386	-0.225	-0.116	-0.114	-0.122	-0.118	-0.173	-0.126	0.011	0
m	1.150	0.680	0.363	0.207	0.010	-0.066	-0.135	-0.173	-0.142	-0.026	0.183
n	-0.400	-0.388	-0.226	-0.110	-0.114	-0.122	-0.146	-0.162	-0.139	-0.014	0
p	1.150	0.684	0.363	0.207	0.014	-0.058	-0.047	-0.126	-0.078	-0.002	0.183
q	-0.400	-0.388	-0.227	-0.108	-0.095	-0.065	-0.054	-0.034	0.009	0.078	0
r	1.150	0.685	0.362	0.344	0.219	0.198	0.184	0.181	0.151	0.148	0.463
s	1.150	0.984	0.763	0.678	0.698	0.598	0.528	0.467	0.281	0.234	0.952
t	1.150	1.150	1.060	0.973	0.850	0.735	0.625	0.547	0.353	0.265	1.165

## 9. Observed Settlements

Figure 11 shows the time-settlement relation for one point in the building. This curve is drawn smoothly among the points representing actual observations, and is believed to be a fair approximation to the actual time-settlement curve. The end of the primary consolidation appears to be about 17 years after the start of construction. The time-settlement curves for the other points were also drawn and had the same general shape. The end of the primary consolidation according to these curves was also found to occur from 16 to 18 years after construction. Hence, 17 years was assumed to correspond to the end of primary consolidation for determining the observed settlements.

The irregularities in the actual time-settlement curve, Fig. 11, may partly be the effects of changes in the level of Lake Michigan. A curve showing the lowest monthly lake level is included at the bottom of Fig. 11.\* Lake level was at the lowest point in history during the period 1922 to 1925. The resulting increase in consolidation stress may have initiated a new phase of primary consolidation, as the settlement readings indicate. However, this increase was disregarded in the graphical construction for determining the observed settlements.

## 10. Discussion of Computed and Observed Settlements

Figure 10 represents the settlement profile showing both computed (dash line) and observed (full line) values. The curve of computed settlements is composed of different bowl-shaped sections, whereas the actual settlements correspond to a rather smooth curve. The dotted portion of the observed settlement curve is the extension of the full line beyond the points where settlement observations were made.

\* Information presented by this curve was obtained from "Report of the Engineering Board of Review of the Sanitary District of Chicago on the Lake Lowering Controversy and a Program of Remedial Measures," Part II, The Technical Base for the Recommendations of the Board of Review, Jan. 23, 1925.

On the whole, the observed and computed settlements are in general agreement, and the order of magnitude of the settlements is the same. The maximum settlement occurs under the tower section in both cases. Furthermore, the maximum differential settlements indicated by both curves are approximately the same. This is a very important fact from the viewpoint of the designing engineer, because the secondary stresses thrown into a building frame are determined by the differential rather than by the average settlement. The discrepancies between the two curves may be partly the result of the assumptions made in computing the stresses, particularly in that the structure was assumed to be flexible and the ground surface free to deform without restraint. The computed settlement curve has the shape of a deep bowl under the tower footing, whereas the actual settlement curve is practically plane and indicates the greatest settlement toward the exterior of the building. The tilt is probably due to the unsymmetrical stress condition beneath the tower due to the relief of stress caused by the basement excavation inside the building. Furthermore, the tower itself is a very rigid unit which could not possibly deflect into the shape of a bowl to the extent indicated by the curve of computed settlements.

In addition, the computation indicates no settlement of the unloaded areas between footings, whereas these points have actually settled. This is caused in part by tough uppermost clay layer A (Fig. 9), which acts to some extent as a mat and distributes the load over the area of the entire building.

In spite of all these discrepancies, the computed settlement curve is a close approximation to the actual settlement curve, and the agreement between them is as good as would be necessary for a designer to judge the adequacy of the foundation.

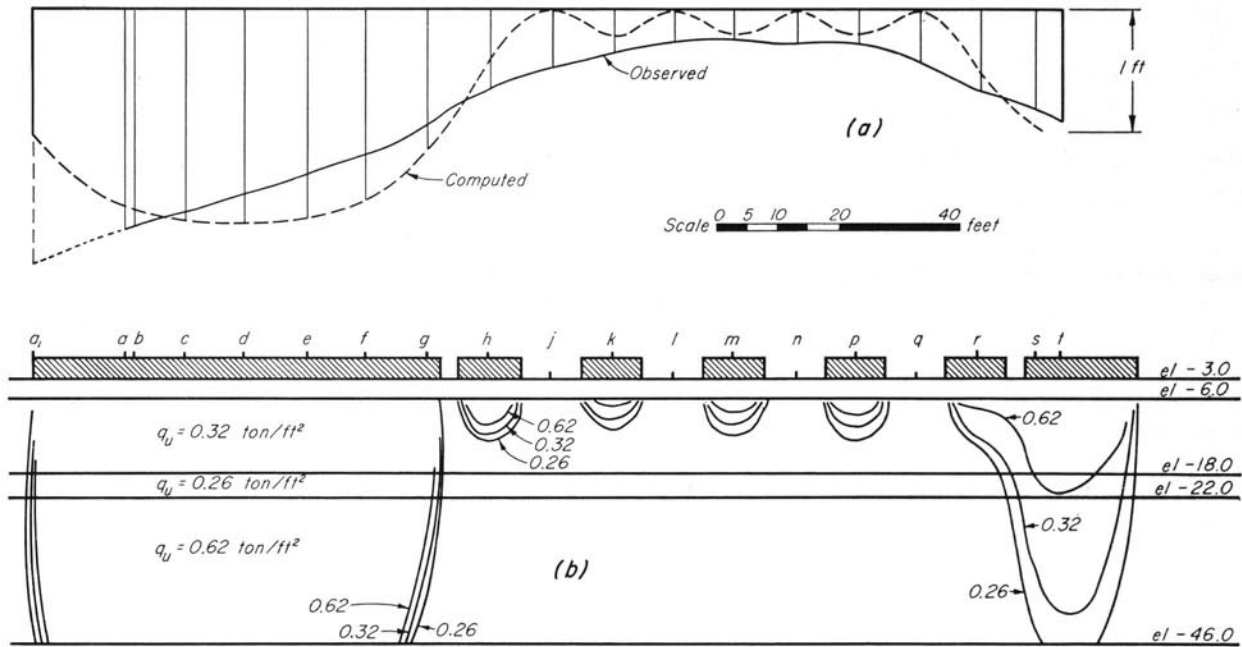


Fig. 10. Comparison for Chicago Auditorium of (a) Computed and Observed Settlements; (b) Net Vertical Pressure in Subsoil and Unconfined Compressive Strength (tons per sq ft)

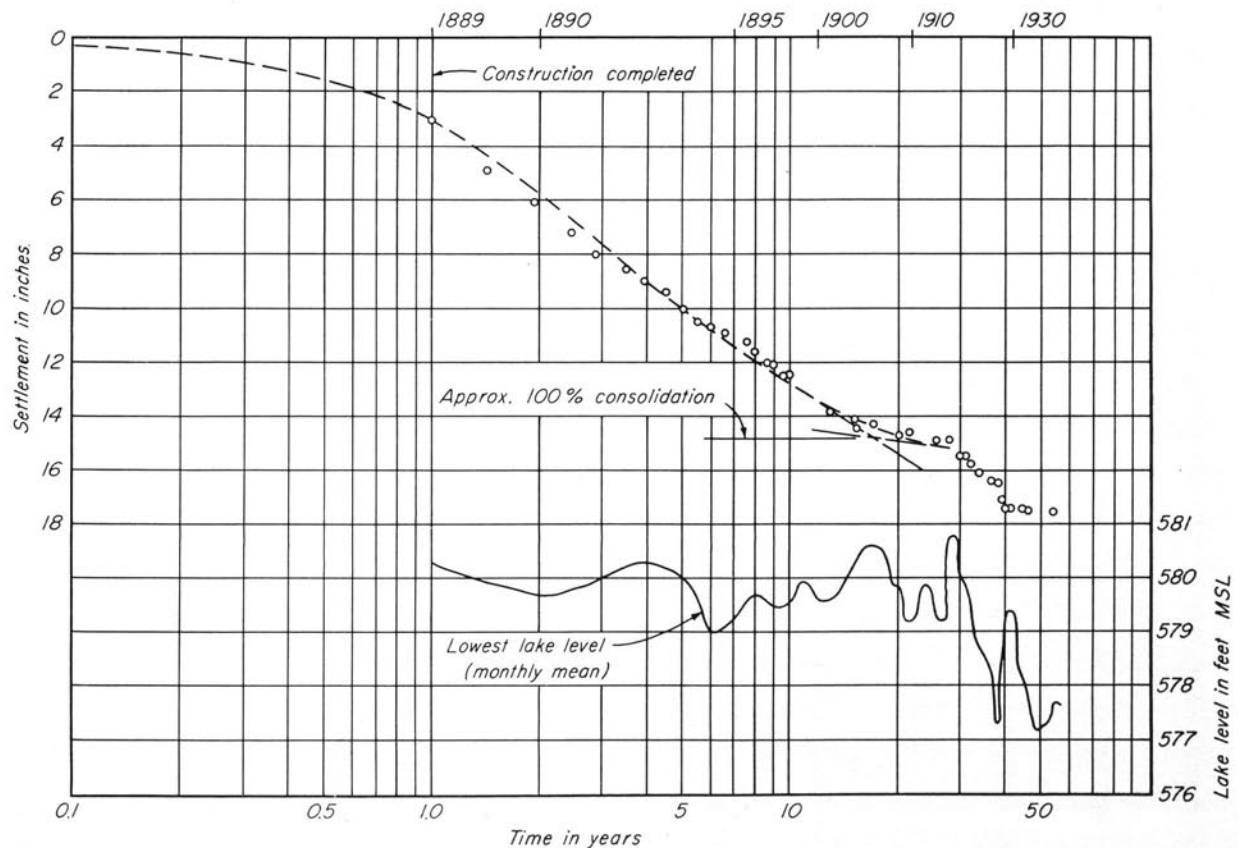


Fig. 11. Time-Settlement Relation for Typical Point of Auditorium

### III. MASONIC TEMPLE

#### 11. Information Available

The Masonic Temple was located on the northeast corner of State and Randolph Sts. It was a steel-frame structure, 113 by 165 ft in plan, and 302 ft high. It was the first 20-story building to be constructed (1892), and for many years was the tallest building in the world. The general appearance is indicated in Fig. 12. In 1940, the structure was demolished.

The structure was founded on spread footings at El. 0.0. The footing plan is shown in Fig. 13. Considerable information about the building was given by its engineer, E. C. Shankland, in a paper, "Steel Skeleton Construction in Chicago," in the



Fig. 12. Masonic Temple

Proceedings of the Institution of Civil Engineers, 1897, part II. In this paper the total dead load on one footing, including the weight of the foundation, was given. The size of the footing was also indicated. From this information the soil pressure was found to be 3070 lb per sq ft. However, according to information furnished by Mr. Shankland to E. L. Corthell for publication in Appendix A of "Allowable Pressures on Deep Foundations" by the latter, the maximum footing pressure was 3200 lb per sq ft.

Construction of the Masonic Temple extended from November 1890 to November 1891. In his 1897 paper, Shankland recorded the results of level readings on six columns, starting in May 1891, and continuing until September 1895. These data are given in Table 4, in which the column numbers are the same as those shown in Fig. 13. Reference is made in the paper to readings taken two years later, but these have not been located.

In writing Mr. Corthell in 1903, Shankland stated that the building had settled a maximum of  $14\frac{1}{2}$  in. from May 1891 to July 1902, and that from July to October 1902 there had been no settlement.

In 1913, a survey of the elevations of the first floor and sidewalks was made by Elmer Clausen, and referred to city datum. Settlement contours based on this survey are shown in Fig. 14. According to Clausen's survey, the settlements in 1913 at points near those reported by Shankland are

Column	1	8	19	26	69	70
Settlement (in.)	$8\frac{3}{8}$	$8\frac{15}{16}$	$11\frac{3}{4}$	$9\frac{1}{4}$	$9\frac{3}{8}$	$9\frac{9}{16}$

The relative magnitudes of these settlements are in agreement with those observed in 1893 and 1895.

Figure 15 shows semi-logarithmic time-settlement curves for the six points, on the assumption that the building was not set above the grade shown on the plans. The plans indicate that the inside grade of the first floor was set 8 in. above the established curb grades, and there appears to be no indication that any additional allowance was made for settlement. The trend of the time-settle-





Fig. 13. Footing Plan of Masonic Temple

ment curve for point 19 indicates that in 1903 the settlement should have been about 11.7 in., whereas Shankland states that the maximum settlement in that year was 14.5 in. Since there is no assurance that point 19 represented the point of maximum settlement in the entire building in 1902, the discrepancy of 2.8 in. does not seem unreasonable. In any event, it is evident that the building could not have been constructed more than this amount above the grade indicated on the plans. It seems most probable that no adjustment in the time-settlement

curves is required to take account of the initial elevations.

Soil conditions are indicated by a Subway boring at State and Randolph Sts.

## 12. Assumptions for Computation

Figure 16 is the log of the boring indicating the soil conditions under the building. The stiff clay layer between El. 0.0 and El. -2.5 was assumed to be incompressible. The seat of settlement was assumed to lie between El. -2.5 and El. -41.0. Also, as shown in Fig. 16, the seat of settlement was assumed to consist of two layers with different water contents. The upper twenty feet, between El. -2.5 and El. -22.5, were assumed to have an average water content of 29.0 percent and the layer underneath an average water content of 24.0 percent. The upper layer (see Fig. 17) was subdivided into four equally thick sublayers (5.0 ft each), and the lower 18.5-ft layer was subdivided into two equally thick layers (9.25 ft each).

Table 4  
Settlement of Selected Columns in Masonic Temple  
According to Shankland (1897)

Column	1	8	19	26	69	70
May 19, 1891	0	0	0	0	0	0
July 9	$\frac{1}{4}$	$\frac{1}{4}$	$\frac{3}{8}$	$\frac{1}{4}$	$\frac{5}{16}$	$\frac{5}{16}$
Aug. 9	$\frac{5}{16}$	$\frac{1}{2}$	$\frac{3}{8}$	$\frac{1}{2}$	$\frac{5}{16}$	$\frac{5}{8}$
Oct. 18	$1\frac{3}{8}$	$1\frac{1}{2}$	$2\frac{1}{8}$	$1\frac{3}{8}$	1	$1\frac{1}{8}$
Jan. 3, 1892	$2\frac{7}{8}$	$2\frac{7}{8}$	$3\frac{3}{8}$	$2\frac{3}{4}$	$2\frac{5}{16}$	$3\frac{1}{4}$
Mar. 14	$3\frac{7}{8}$	$4\frac{1}{4}$	$5\frac{1}{2}$	4	$3\frac{3}{8}$	$4\frac{3}{8}$
Apr. 26	$4\frac{1}{4}$	$4\frac{3}{4}$	$6\frac{3}{8}$	$4\frac{1}{2}$	$4\frac{1}{4}$	$4\frac{7}{8}$
July 7	$4\frac{1}{2}$	$5\frac{3}{8}$	7	5	5	$5\frac{3}{8}$
Nov. 4	5	6	8	$5\frac{3}{4}$	$5\frac{1}{8}$	$6\frac{3}{8}$
Oct. 20, 1893	$6\frac{7}{16}$	$7\frac{1}{16}$	$9\frac{7}{16}$	7	$7\frac{3}{8}$	$7\frac{7}{8}$
Sept. 18, 1895	$7\frac{7}{8}$	$8\frac{9}{16}$	11	$8\frac{9}{16}$	...	...

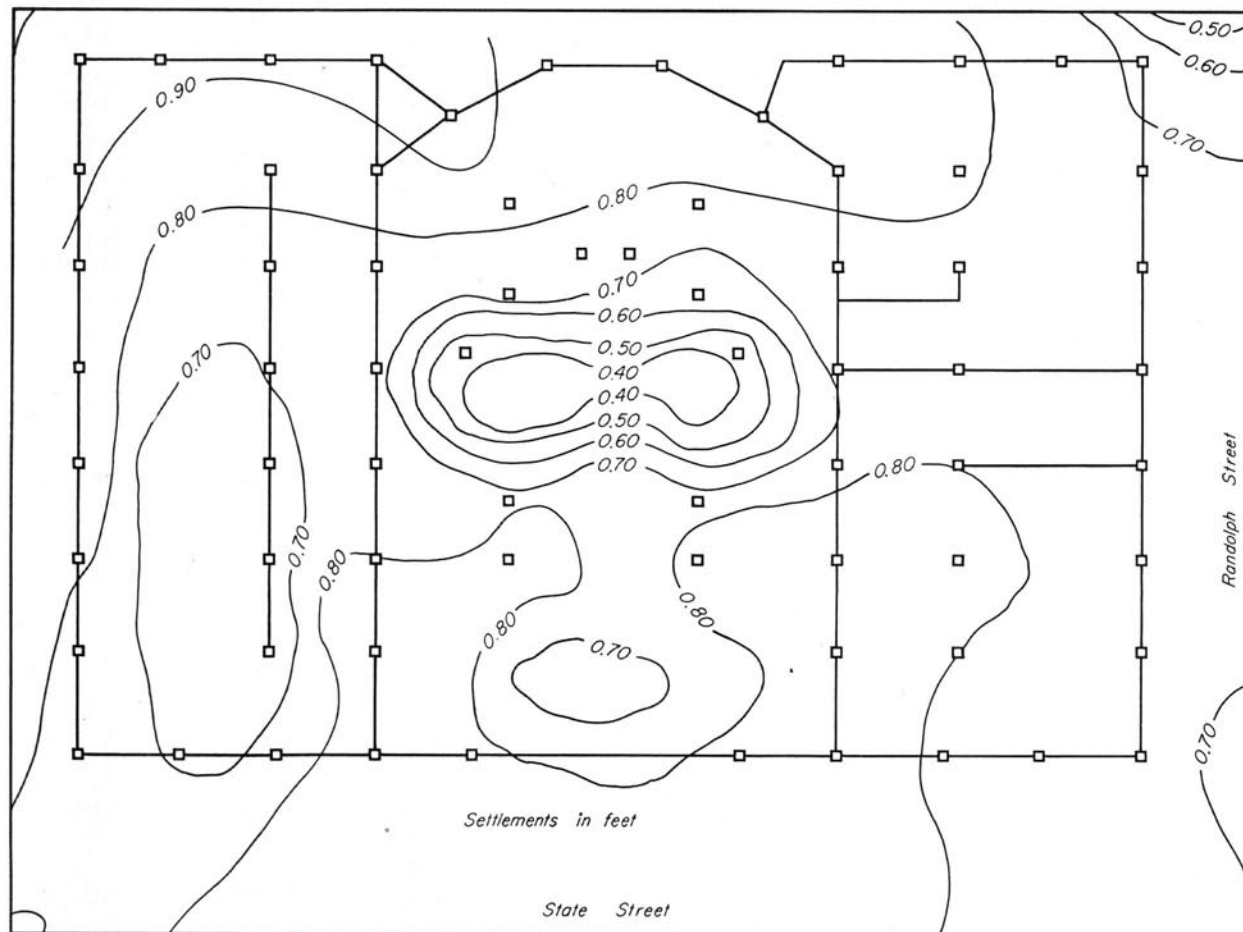


Fig. 14. Settlements of Masonic Temple in 1913

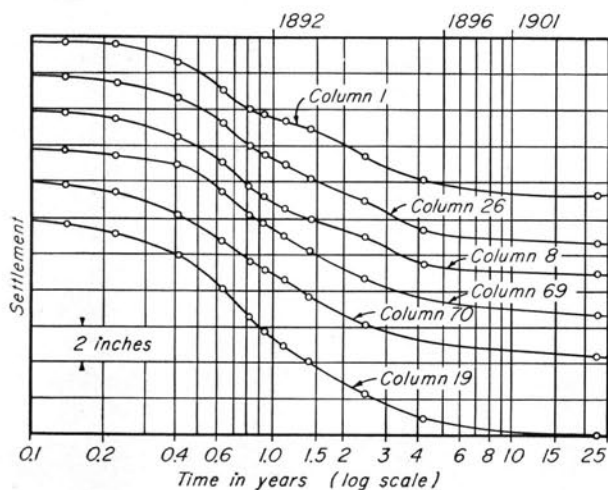


Fig. 15. Time-Settlement Curves for Points of Masonic Temple

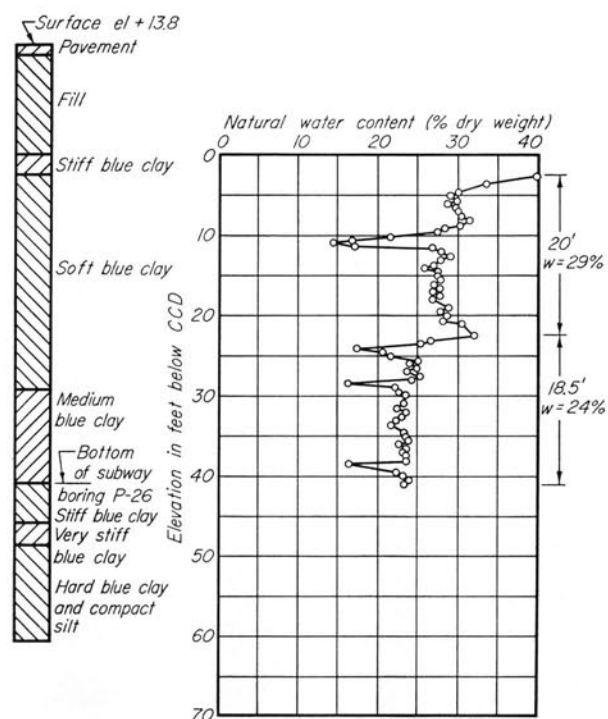


Fig. 16. Log of Boring near Masonic Temple

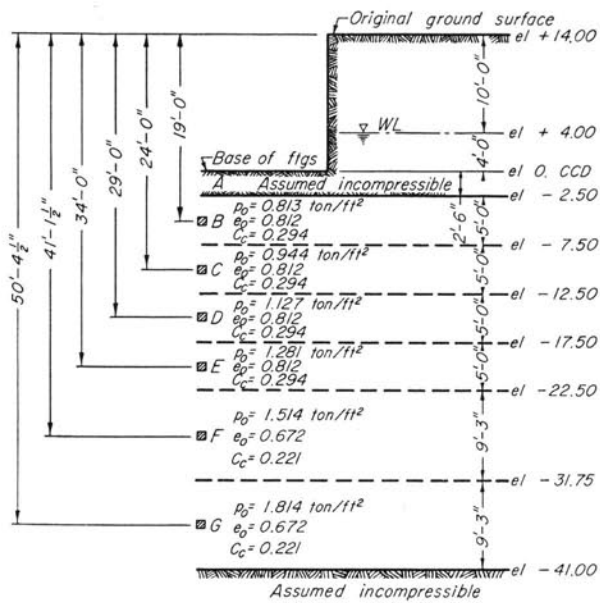


Fig. 17. Data for Computation of Settlement of Masonic Temple

Settlements were computed for 50 points within the area of the building and 10 points on the four curb walls. The footing plan in Fig. 13 shows the location of these points. The numbers of the columns are also indicated. Each point was chosen at the center of gravity of a footing, where the resultant of the column loads was assumed to act. The soil pressure used for stress computation was 3100 lb per sq ft. Stresses were computed at six different depths and settlements were computed for each point. Table 5 gives the final stresses as well as the computed settlements.

### 13. Observed Settlements

Figure 15 shows the time-settlement relations for six points in the building. On the basis of these curves, the primary settlements for the building were found to be 94.7 percent of the total observed settlements in 1913.

### 14. Discussion of Computed and Observed Settlements

The contours of computed settlement are shown in Fig. 18. The patterns of the computed contours and those of the actual (total) settlement shown in Fig. 14 are reasonably similar. The maximum as well as the minimum settlements occur at about the same locations in the building. However, contours of computed settlement show no settlement at all in the center of the building, whereas this part of the building actually settled more than 3½ in. Since the center section is the most lightly loaded area (see footing plan in Fig. 13), the computed stresses are not of sufficient magnitude to cause

Table 5  
Net Vertical Pressures under Masonic Temple  
and Computed Settlements

Point	5	10	15	20	27.13	31.75	Settlement (ft)
1	0.691	0.402	0.240	0.134	0.112	0.099	0.530
2/3	0.745	0.556	0.417	0.309	0.231	0.215	0.733
4/5	0.754	0.550	0.404	0.311	0.238	0.208	0.714
6/7	0.745	0.555	0.412	0.308	0.228	0.198	0.698
8	0.697	0.476	0.277	0.181	0.145	0.147	0.580
9/10	0.711	0.515	0.397	0.328	0.238	0.212	0.702
11/12	0.745	0.565	0.432	0.374	0.291	0.229	0.752
13/14	0.687	0.424	0.276	0.234	0.194	0.183	0.607
15/16	0.752	0.662	0.520	0.487	0.398	0.349	0.881
17/18	0.736	0.583	0.464	0.399	0.314	0.267	0.776
19	0.635	0.386	0.228	0.189	0.196	0.106	0.524
20/21	0.740	0.542	0.394	0.348	0.222	0.144	0.609
22/23	0.742	0.550	0.398	0.295	0.228	0.193	0.703
24/25	0.740	0.550	0.419	0.301	0.235	0.185	0.704
26	0.671	0.428	0.273	0.158	0.110	0.108	0.535
27/28	0.723	0.489	0.344	0.232	0.173	0.127	0.630
29/30/77	0.754	0.639	0.502	0.373	0.255	0.189	0.762
79	-0.375	-0.331	-0.227	-0.120	-0.019	0.005	0.086
33/34/78	0.754	0.640	0.502	0.375	0.269	0.241	0.785
35/36	0.726	0.513	0.346	0.254	0.179	0.147	0.642
37	0.668	0.467	0.374	0.339	0.282	0.249	0.700
38	0.656	0.444	0.355	0.322	0.281	0.242	0.674
39	0.648	0.418	0.310	0.282	0.261	0.219	0.638
40	0.671	0.448	0.323	0.338	0.257	0.240	0.666
41	0.679	0.486	0.371	0.337	0.298	0.251	0.707
42	0.706	0.488	0.427	0.381	0.321	0.279	0.751
43/58	0.758	0.683	0.553	0.492	0.360	0.316	0.878
44	0.682	0.478	0.369	0.305	0.263	0.209	0.675
60/61	0.417	0.158	0.116	0.142	0.219	0.138	0.373
45/62	0.696	0.552	0.342	0.300	0.261	0.221	0.688
46/63/64	0.759	0.700	0.560	0.500	0.396	0.353	0.904
47/48	0.712	0.537	0.469	0.406	0.348	0.315	0.792
49	0.644	0.414	0.306	0.287	0.248	0.235	0.642
50	0.654	0.393	0.296	0.266	0.228	0.220	0.615
51	0.678	0.428	0.347	0.308	0.247	0.231	0.669
52	0.635	0.418	0.358	0.308	0.260	0.232	0.662
53/66	0.748	0.612	0.481	0.435	0.328	0.262	0.809
54/55	0.758	0.614	0.509	0.443	0.329	0.281	0.826
56	0.602	0.385	0.301	0.260	0.260	0.246	0.621
57	0.634	0.428	0.318	0.277	0.257	0.248	0.643
59	0.573	0.304	0.194	0.167	0.157	0.132	0.480
67/70	0.749	0.518	0.343	0.260	0.183	0.127	0.643
68/69	0.749	0.522	0.349	0.247	0.196	0.167	0.662
65	0.570	0.324	0.259	0.242	0.227	0.223	0.564
71	-0.229	-0.058	0.033	0.096	0.143	0.147	0.125
72	-0.222	-0.111	0.007	0.072	0.152	0.157	0.116
73	-0.280	-0.174	-0.120	-0.009	0.105	0.203	0.099
74	-0.238	-0.152	-0.081	0.010	0.088	0.123	0.067
75	-0.356	-0.363	-0.262	-0.151	0.058	0.064	0.032
76	-0.349	-0.381	-0.187	-0.079	0.064	0.099	0.071
80	0.260	-0.083	-0.108	-0.085	-0.038	-0.019	0.099
81	0.170	0.085	0.020	0.086	0.091	0.099	0.191
82	0.205	-0.089	-0.104	-0.071	-0.018	0.001	0.086
83	0.177	0.115	0.113	0.149	0.154	0.161	0.289
84	0.170	-0.063	-0.077	-0.056	-0.029	-0.002	0.072
85	0.140	0.050	0.045	0.093	0.134	0.124	0.180
86	0.200	0.101	0.136	0.144	0.133	0.133	0.277
87	0.220	0.206	0.193	0.178	0.153	0.157	0.357
88	0.200	0.136	0.147	0.134	0.137	0.110	0.277
89	0.223	0.210	0.207	0.185	0.156	0.149	0.364

appreciable settlement. However, in the computation of stresses the building was assumed to be perfectly flexible, and the action of the stiff clay crust (layer A in Fig. 17) as a mat was disregarded. Both these assumptions would tend to make the computed settlements in the central part of the building too small.

Figures 19 and 20 contain settlement profiles showing the computed and actual settlement at various sections through the building (Fig. 18). The two east-west sections (1-1 and 2-2) indicate good agreement in the magnitude as well as in the pattern of the settlement. The same general agreement can also be seen in the north-south section 4-4. Section 3-3 indicates the distinctive lack of settlement shown by both the computed and actual settlement profiles at the center of the building.

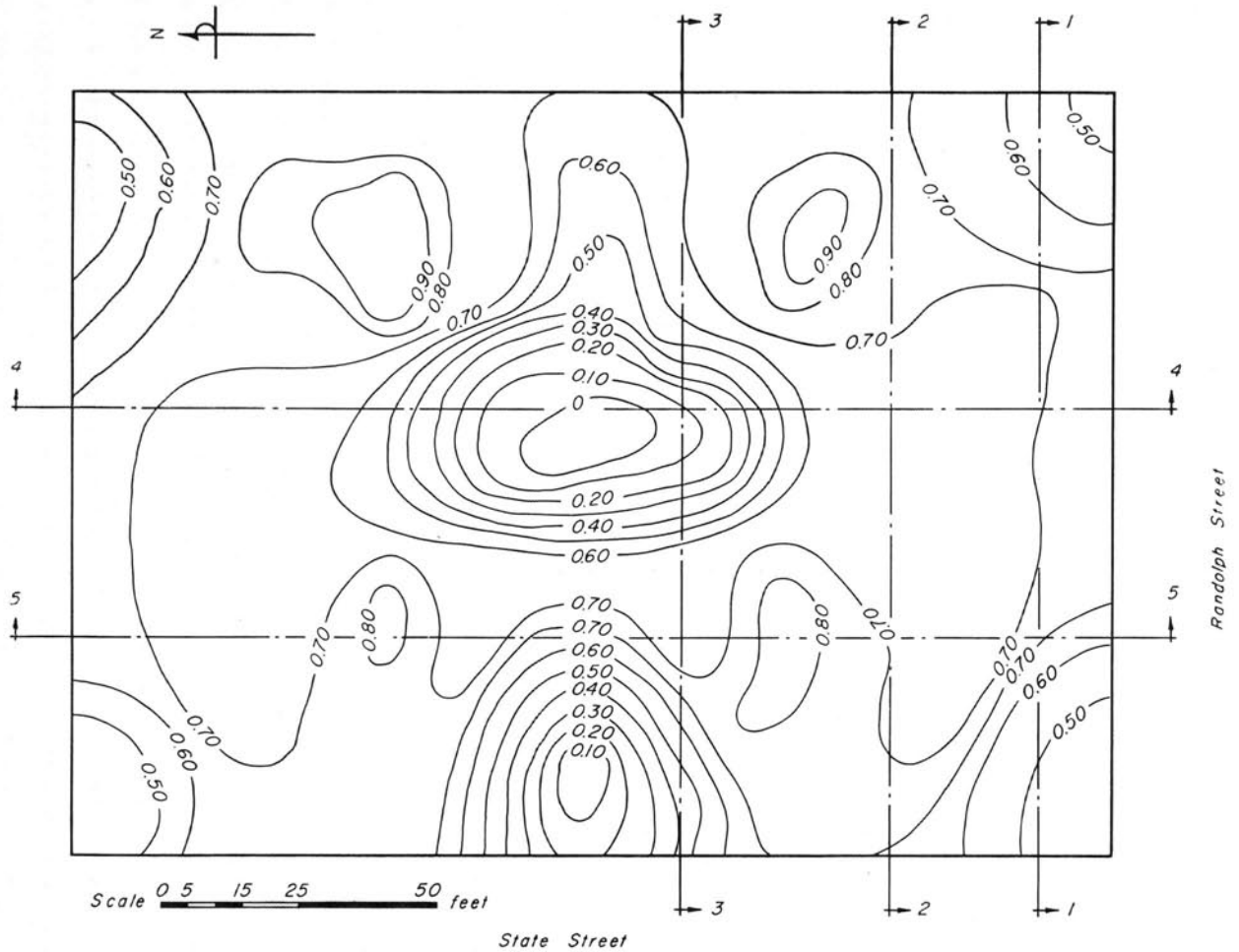


Fig. 18. Computed Primary Settlements of Masonic Temple (ft)

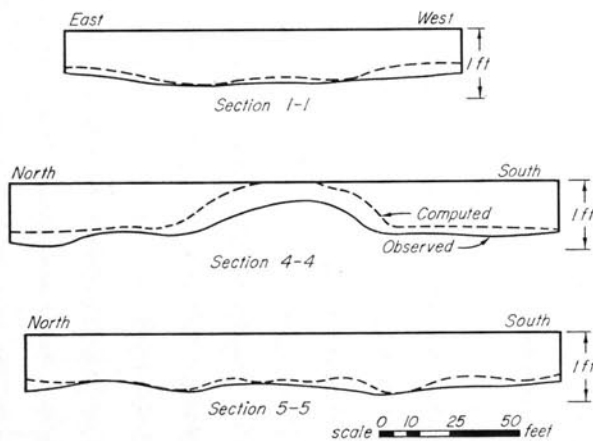


Fig. 19. Comparison of Computed and Observed Settlements of Sections through Masonic Temple

From these comparisons it can be concluded that the computed settlements agree quite satisfactorily with the observed ones.

Figure 20 shows the actual and computed settlements along sections 2-2 and 3-3, as well as the relation between the stresses and the strength of the soil beneath the building at these sections. The strengths are those determined in unconfined compression tests on 2-in. samples taken in thin-walled tubes.

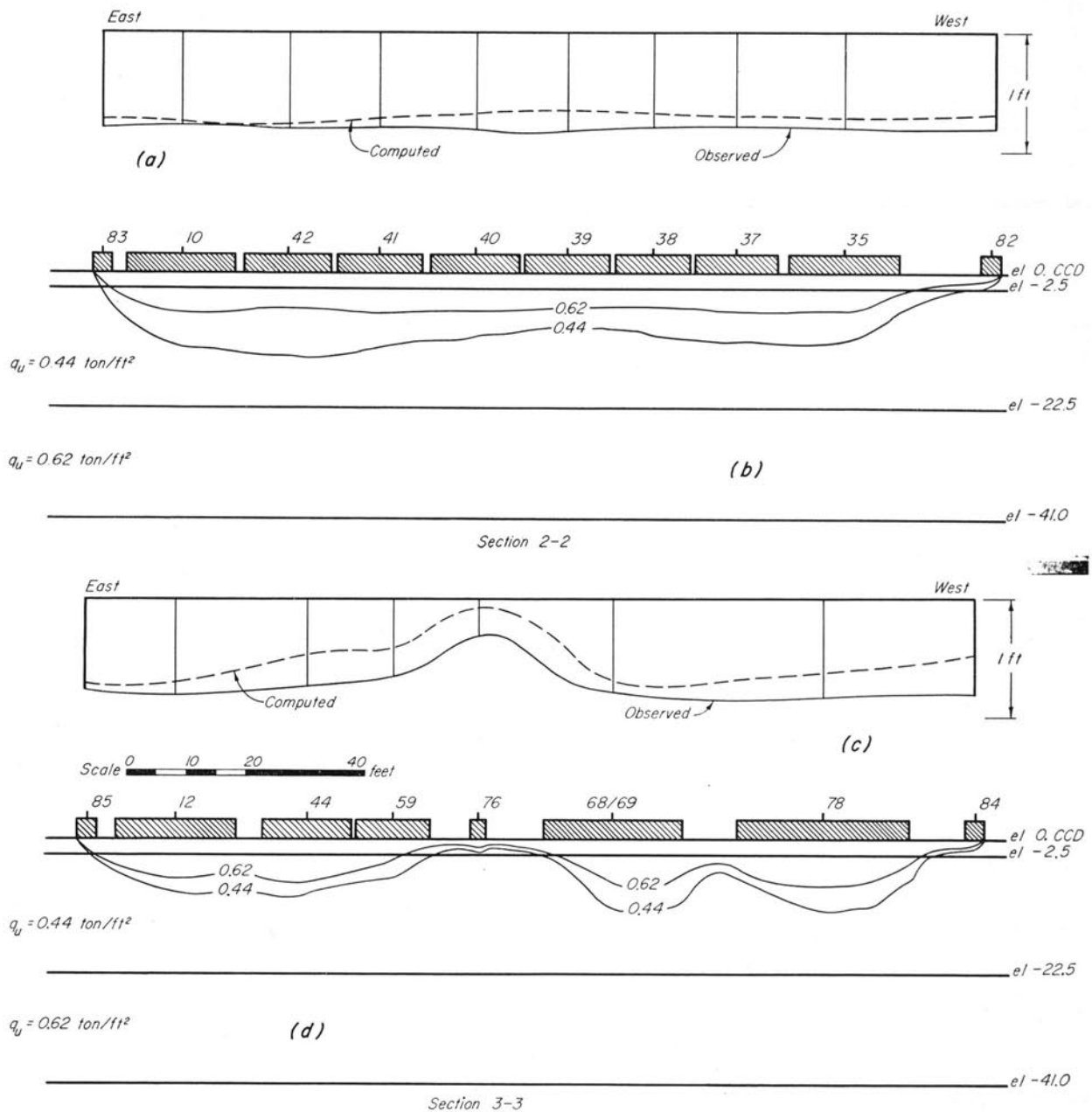


Fig. 20. Comparison for Two Sections through Masonic Temple of (a and c) Computed and Observed Settlements; (b and d) Net Vertical Pressure in Subsoil and Unconfined Compressive Strength (tons per sq ft)



## IV. MONADNOCK BLOCK

### 15. Information Available

The Monadnock Block, Fig. 21, is a 16-story wall-bearing masonry building 66 by 398 ft in plan, occupying the block bounded by Dearborn, Federal and Van Buren Sts and Jackson Blvd. The North Section, 199 ft long, was completed in 1891, and the South Section in 1892. The foundation consists of concentrically loaded footings. Wall footings project beyond the building lines. The footings were originally founded approximately at El. +2.0 and the basement floor was backfilled to about El. +6.0. A curb wall surrounds the entire basement. The general location of the footings and curb wall is shown in Fig. 22.

In 1939, a design was prepared by Holabird and Root for underpinning the east line of columns prior to constructing the Dearborn St Subway. In connection with this design, a careful estimate was made of the dead load supported by the individual columns. From this information, the soil pressure due to dead load was found to be about 3900 lb per sq ft. According to O. Guthrie,\* the maximum pressure was 3750 lb per sq ft, presumably including live load.

Settlement information is contained in the results of surveys made on May 16, 1892, by Alexander Hill; on June 26 and 27, 1900, by Emil Rudolph; in June, 1902, by Emil Rudolph; and on December 21, 1914, by the Chicago Guarantee Survey Company. These surveys consisted principally of determining the elevations of unidentified points on the main floor, the sidewalks, and the curbs. Levels on selected points were taken during the period 1911-1916 by the Chicago Guarantee Survey Company, and on other points during the period from November 1924 to November 1927 when the Standard Club, on the east side of Dearborn St, was under construction. All these levels were referred to Chicago City Datum.

Established legal curb grade on all sides of the structure was El. +14.0. Inside grade on Dearborn St was 14 ft, 5½ in., corresponding to the legal sidewalk slope of 1⅓ in. per ft. However, according to John M. Ewen,† the North Section was es-

tablished about 8 in. high to allow for settlement. This value presumably refers to inside grade and indicates that the original first floor elevation was at 15 ft, 1½ in. Evidence in support of this statement is found in the facts that the northeast corner of the building was still 4½ in. above inside grade in the Alexander Hill survey of 1892, whereas Guthrie states that in 1892 the settlement in some parts of the building was 5 in. Since the corner undoubtedly settled less than other points in the structure, the structure must have been set something less than 9½ in. high. In 1900, the corner settlement was 3½ in. less than the maximum



Fig. 21. Monadnock Block

\* Eng. News, Vol. 92, ii, p. 345.

† Journ. W.S.E., 1905, p. 687.

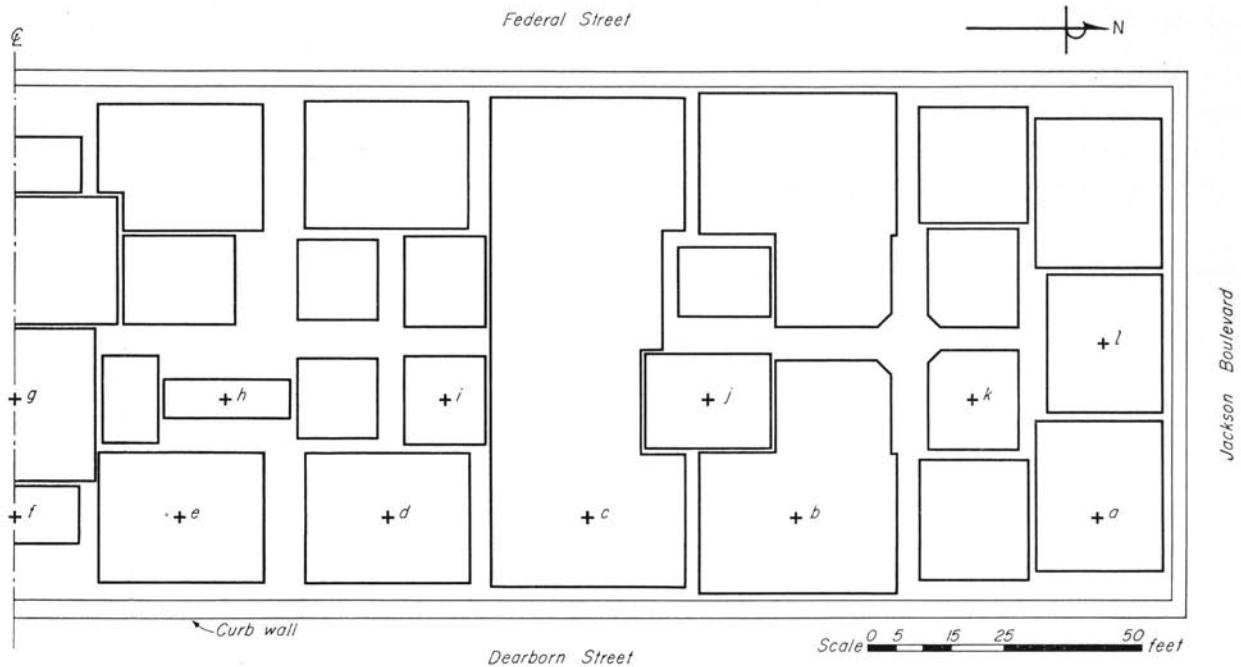


Fig. 22. Footing Plan of North Half of Monadnock Block

recorded value; since this differential settlement undoubtedly increased from 1892 to 1900, the building must have been set high at least as much as  $9\frac{15}{16}$  in. minus  $3\frac{13}{16}$  in. or  $6\frac{1}{8}$  in. The value 8 in. seems very reasonable in view of these two independently determined limits.

The June 1900 survey plot shows a profile of sidewalk levels along the building line. Since the sidewalk was supported by brackets on the masonry piers at the building line, the settlement represented by this profile also represents the settlement of the east side of the building. The results are reproduced in Fig. 23. The survey of 1902 provided similar data for the settlement of the west side of the building. The 1916 survey plot shows a number of floor elevations, but too few to construct a profile for comparison with 1900. In 1940, the brackets that supported the original sidewalk slabs were about 21 in. below their probable initial position. At this time, the east wall was underpinned and settlement ceased.

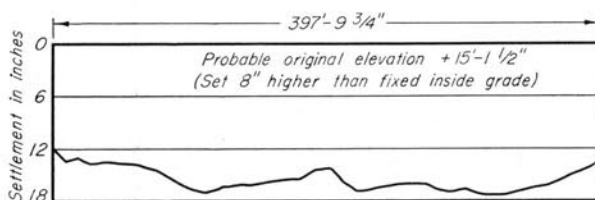


Fig. 23. Settlement of East Side of Monadnock Block in 1900

By taking into account all the available data, a time-settlement curve was constructed by V. O. McClurg and offered as court evidence in 1943. A semi-logarithmic plot of this curve is shown in Fig. 24. It represents the trend of the settlement for the North Section, and probably is a very reliable representation of the settlement of the north-east corner of the building.

The survey plot of 1902 indicates that the bench mark used in previous surveys was in error by  $\frac{3}{8}$  in. Hence, the points on the time-settlement curve are certainly not accurate within less than this amount, and very probably are as much as an inch in error considering the fact that no definite reference points were established.

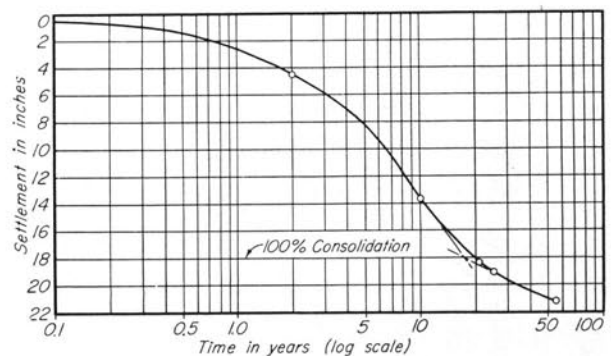


Fig. 24. Time-Settlement Relation for Point on Monadnock Block



The soil conditions within the compressible strata below the building are indicated by three Subway boring logs, all located on Dearborn St. One boring is about 100 ft south of the south building line of the Monadnock Block, the second is slightly north of the middle of the structure, and the third is about 100 ft north of the north building line.

### 16. Assumptions for Computation

The original ground surface was assumed to be at El. +14.0. The building was assumed to have a symmetrical footing plan about both axes, and settlements were computed for twelve points in one quarter of the area of the building. The other three quarters of the building were assumed to have settled symmetrically about the axes.

The seat of settlement was assumed to lie between El. -3.0 and El. -46.0 (see Fig. 25 showing the soil conditions under the building). The compressible part of the deposit was divided into two layers with different water contents. The layer between El. -3.0 and El. -15.0 was assumed to have a constant water content of 27.0 percent and was subdivided into two 6-ft layers, B and C, Fig. 26. The layer between El. -15 and El. -46.0 was assumed to have a water content of 23.0 percent and was divided into four layers 7.75 ft thick. Stresses were computed at six different depths and are shown in Table 6 with the computed settlements under each point.

### 17. Observed Settlement

Figure 24 shows the time-settlement curve referred to in Section 15, and Fig. 23 represents the settlement profile made of the east wall by Emil Rudolph in 1900. The settlements given by Rudolph were increased by the ratio 1.27, determined on the basis of the time-settlement curve, to obtain the profile of observed settlements (100 percent consolidation) for comparison with the computed settlements. The profiles of computed and observed settlements and the stresses in the soil corresponding to the unconfined compressive strength of the compressible layers are shown in Fig. 28.

### 18. Discussion of Computed and Observed Settlements

Figure 27 shows a set of computed settlement contours. Since there are no actual settlement contours available, the computed settlement contours cannot be used for comparison. However, they show that the settlement of the entire building is one foot or more. They further indicate that differential settlements are not large. Since there is no extreme

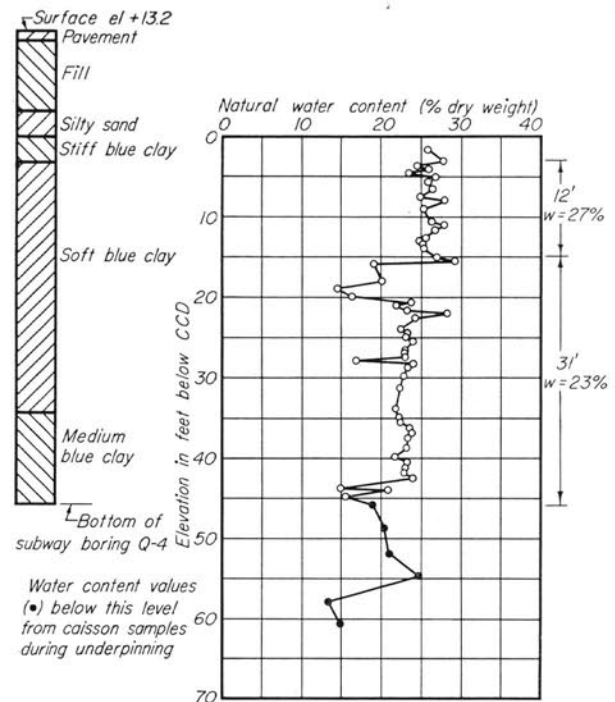


Fig. 25. Log of Boring near Monadnock Block

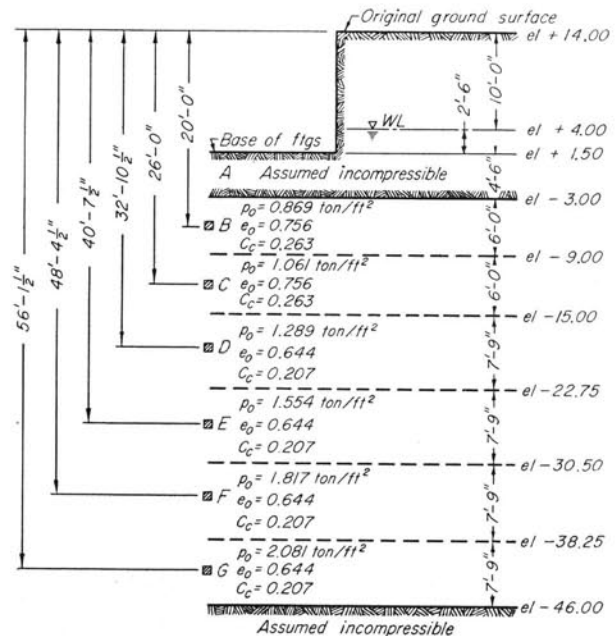


Fig. 26. Data for Computation of Settlement of Monadnock Block

variation in the distribution of load in the building area, greater differential settlements would not be expected.

Figure 28 shows settlement profiles along the east wall. It indicates both the computed and actual values of settlement, as well as the magnitude of the stresses in the soil compared to the unconfined

Table 6  
Net Vertical Pressures under Monadnock Block  
and Computed Settlements

Point	7.5	13.5	20.4	28.1	35.88	43.62	Settlement (ft)
a	1.087	0.897	0.746	0.577	0.510	0.433	1.075
b	1.192	1.081	0.937	0.804	0.713	0.622	1.276
c	1.189	1.077	0.941	0.837	0.735	0.645	1.292
d	1.160	0.950	0.765	0.670	0.597	0.529	1.151
e	1.147	0.920	0.719	0.633	0.555	0.481	1.110
f	0.937	0.751	0.644	0.641	0.582	0.514	1.032
g	1.195	1.136	1.001	0.873	0.782	0.687	1.331
h	0.498	0.433	0.442	0.515	0.555	0.529	0.800
i	1.003	0.854	0.789	0.750	0.718	0.656	1.162
j	1.104	0.983	0.948	0.885	0.805	0.747	1.295
k	0.970	0.752	0.721	0.671	0.628	0.590	1.085
l	1.093	0.930	0.807	0.679	0.589	0.511	0.141

compressive strengths of the two compressible layers. The two settlement curves are similar and indicate almost the same differential settlements, but the observed settlements are about 30 percent greater than the computed ones. As much as about 10 percent of this difference may possibly be attributed to the fact that the layers below El. -46.0 are highly stressed and may have contributed some settlement. The balance, at least 20 percent, must be accounted for otherwise.

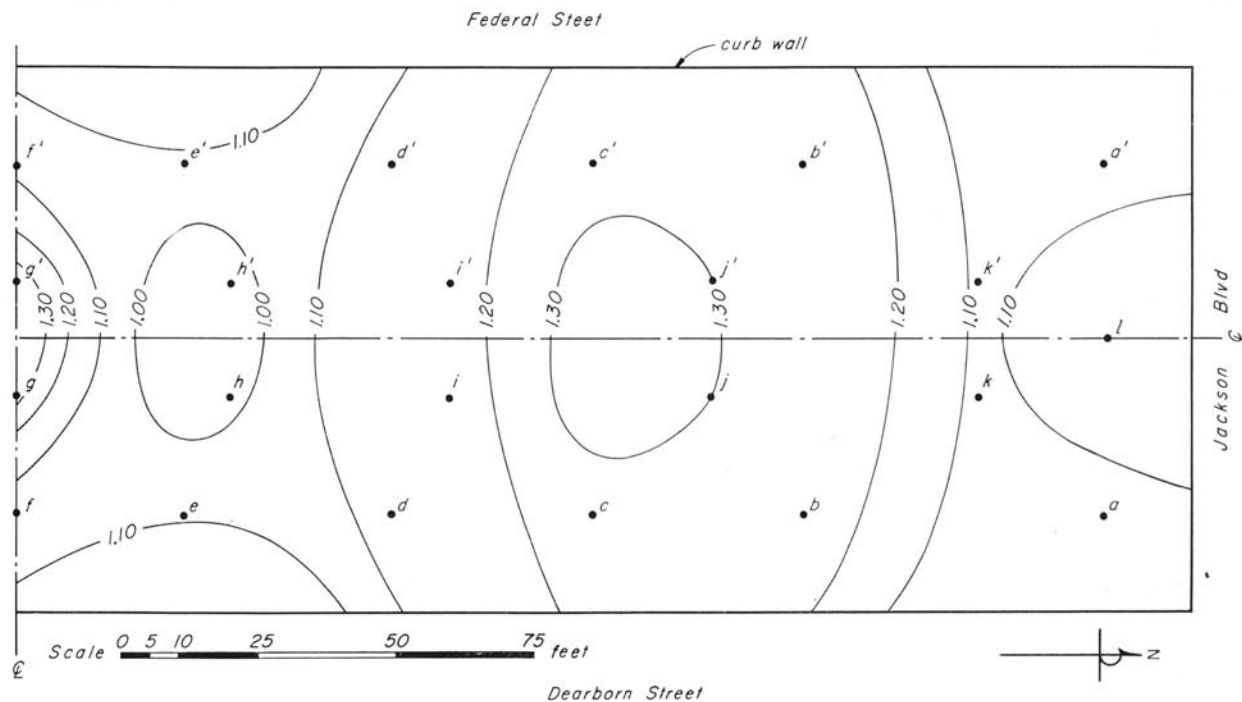


Fig. 27. Computed Primary Settlement of Monadnock Block (ft)

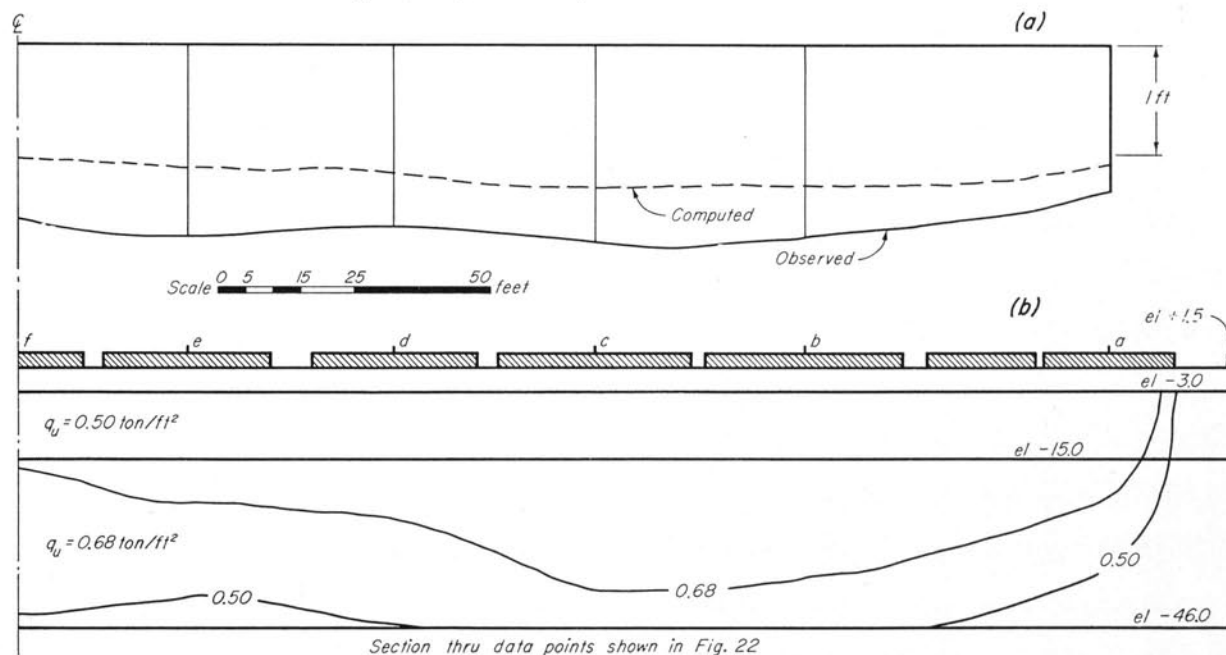


Fig. 28. Comparison for Monadnock Block of (a) Computed and Observed Settlements; (b) Net Vertical Pressure in Subsoil and Unconfined Compressive Strength (tons per sq ft)

## V. OLD BOARD OF TRADE

### 19. Information Available

The Old Board of Trade occupied the northern 225 ft of the block bounded by Sherman St, Jackson Blvd, La Salle St and Van Buren St. It was 225 by 174 ft in plan within building lines. It consisted of a front portion 140 ft high, in which the trading hall was located, and a rear portion 160 ft high, used for offices. In the front, on Jackson Blvd, rose a tower 303 ft high; the lower 225 ft were of granite masonry and the upper 78 ft of cast iron. A general view of the building is shown in Fig. 29, and a map showing its surroundings in Fig. 30.

No footing plan is known to exist; however, a table has been preserved by Mr. R. C. Smith that gives the load on each footing and the area of the footing. This table, prepared by Gen. Wm. Sooy Smith in 1893, refers to footings numbered as



Fig. 29. Old Chicago Board of Trade as Originally Built

shown in Fig. 31. These numbers correspond to those used by R. C. Smith in subsequent settlement reports, and may be assumed to have the same meaning.

In 1894, the American Architect and Building News reported a statement (Feb. 24, p. 92) of the architect that the building foundations rested on

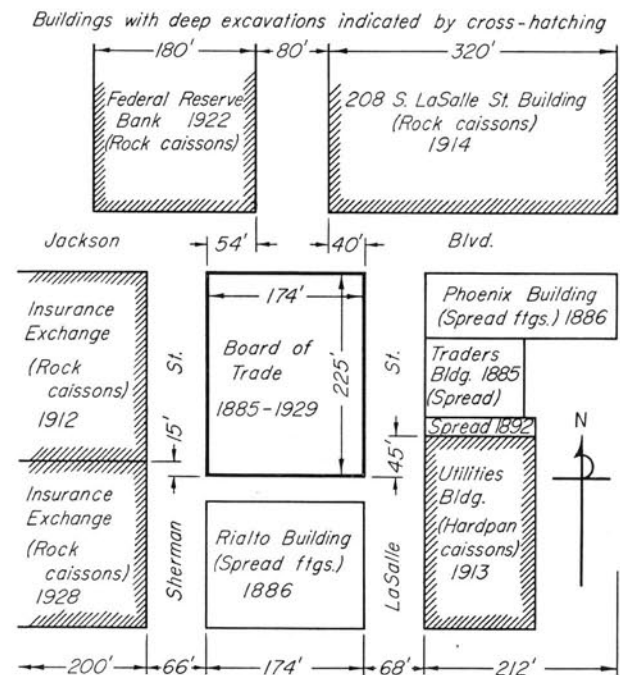


Fig. 30. History of Foundation Construction near Board of Trade

a bed of concrete, prepared as for the foundations of more modern buildings (on rail or beam grillages) except that wooden beams were embedded in the concrete. Studies of photographs taken during demolition (Figs. 32 and 33) in 1928 suggest that the wall footings were continuous, and also indicate the probable location of footings that carried the loads from individual columns. Unfortunately, no photograph was taken at the time the old footings were exposed. It is the recollection of those present during the demolition that the footings were founded near El. 0, and that the basement floor was approximately at El. +3.0.

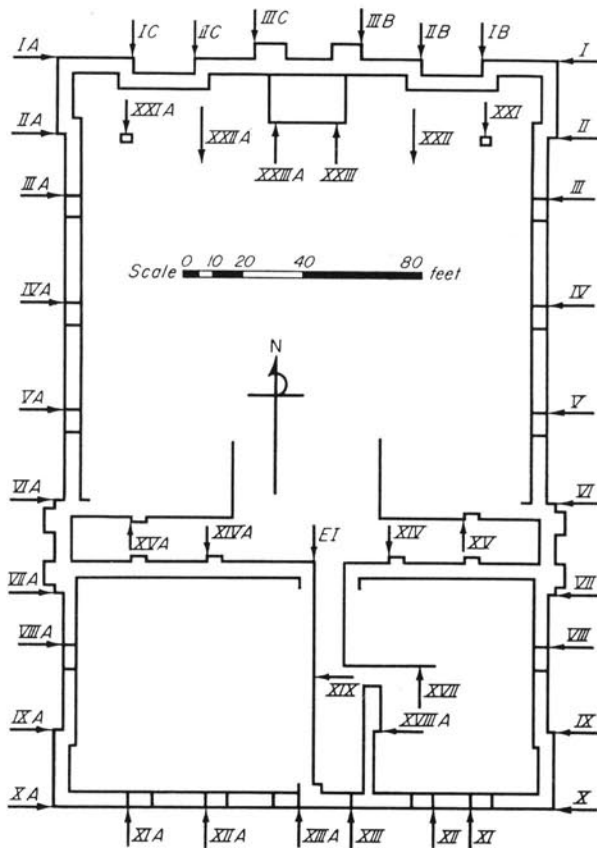


Fig. 31. Location of Reference Points for Settlement of Board of Trade

The structure was built in 1884. By 1895, the tower portion had settled excessively and the upper part of the tower was removed (Fig. 34). The structure then stood until 1928 when it was demolished. In 1889, a set of settlement levels was taken by E. Rudolph on the walls of the structure facing the three streets. Between 1889 and 1895, there is no record of any levels. In 1895, new reference marks seem to have been established and all subsequent levels are reported in terms of the elevations in 1895. From 1895 to 1904, levels were taken by an unknown party; from 1904 to 1919 by Burton J. Ashley; and from 1919 to 1928 by R. C. Smith, who supplied the preceding information. Mr. Smith also had access at one time to a report, since lost, which stated that from 1895 to 1909 the settlement in the Board Room had increased by  $2\frac{1}{4}$  to 3 in. and in the office portion by  $1\frac{1}{4}$  to  $1\frac{1}{2}$  in. The total settlement in 1895 was said to range from  $8\frac{7}{8}$  to 17 in., and in 1909 from  $10\frac{1}{8}$  to 20 in.

Numerous structures were built adjacent to the building and undoubtedly contributed to its settlement (see Fig. 30). However, the first such external

cause of settlement was probably the Insurance Exchange Building, constructed in 1912. The observed settlements throughout the life of the building are given in Table 7, and shown graphically (from 1884 to 1889) in Fig. 35.

A subway boring is located at the corner of Jackson Blvd and La Salle St, directly across La Salle St from the Old Board of Trade.

## 20. Assumptions for Computation

Since there was no footing plan available, Table 8, which gives the loads and soil pressures for each footing, was used in conjunction with the first floor plan, Fig. 31, to prepare an approximate foundation plan. This was done as follows: the points shown in Fig. 31 were assumed to indicate the locations of columns, and each column footing was assumed to consist of that section of a continuous wall footing extending half the distance to the adjacent columns. By providing the proper area (Table 8) for each footing the width of the footing was determined, and small adjustments were made to avoid unreasonable changes in the width of the continuous footings. This procedure gave a very close approximation to the value of the soil pressure calculated by Gen. Sooy Smith under each



Fig. 32. Early Stage of Demolition of Board of Trade





Fig. 33. Masonry Walls of Board of Trade at Ground Floor

footing. The average soil pressure, calculated from Gen. Sooy Smith's table, was 3.34 tons per sq ft, whereas the average pressure obtained from the prepared footing plan was 3.35 tons per sq ft. This justifies the statement that the prepared footing plan is a close approximation. In order to agree with photographs (Figs. 32 and 33) taken during demolition, several small (10 by 10 ft) column footings were added in the office portion of the building (see Fig. 36).

Inasmuch as the unit pressures under the various columns were different, a stress computation made in the usual manner would have been extremely laborious. Therefore each footing was replaced by a circle that would carry the same loads with a uniform soil pressure of 3.35 tons per sq ft under all footings, and thus the stress computations were simplified. The error resulting from this simplification is very small, and can be considered negligible.

The base of footings was assumed to be at El. 0.0. The original ground surface was taken as

El. +14.0. Figure 37 shows the soil conditions under the building. The seat of settlement was assumed to lie between El. -2.5 and El. -41.5. The variation of water content throughout the entire depth to El. -41.5 was very small (see Fig. 37). Therefore, a constant water content of 22.5 percent was assumed for the entire seat of settlement. The compressible clay deposit was subdivided into five layers, the upper two each 6 ft thick and the remaining three each 9 ft. Figure 38 shows all the layers with their properties. Finally, the building was assumed to be symmetrical about its north-south axis. Therefore, settlements had to be computed for only one side of the building. Computed stresses and settlements are given in Table. 9.

## 21. Observed Settlements

The available information is not sufficiently complete to permit drawing a time-settlement curve for any point in the building, because settlement observations were made to determine only the in-



Fig. 34. Old Chicago Board of Trade after Removal of Tower

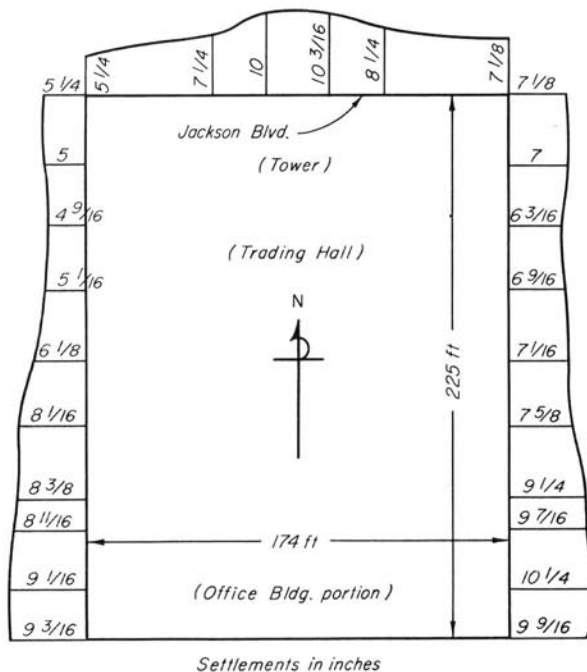


Fig. 35. Settlement of Board of Trade, 1884-1889

Table 7  
Old Board of Trade Building:  
Settlements of Piers and Footings

Points	Settlements in feet				
	1884 to 1889	1895 to 1904	1904 to 1913	1913 to 1919	1919 to 1928
I	.594	.083	.004	.008	0.088
II	.583	.083	.077	.036	0.101
III	.516	.059	.089	.037	0.092
IV	.547	.072	.084	.028	0.067
V	.589	.115	.080	.036	0.075
VI	.635	.122	.084	.049	0.110
VII	.771	.122	.071	.047	0.089
VIII	.787	.122	.087	.057	0.073
IX	.859	.122	.083	.050	0.084
X	.797	.156	.087	.061	0.051
XI		.122	.074	.059	0.043
XII		.122	.078	.060	0.019
XIII		.167	.087	.066	0.039
XIIIA		.167	.087	.051	0.061
XIIA		.155	.089	.054	0.064
XIA		.177	.097	.044	0.035
XA	.766	.156	.111	.049	0.054
IXA	.755	.156	.120	.045	0.080
VIIIA	.724	.145	.136	.047	0.079
VIIA	.698	.157	.141	.038	0.085
VIA	.672	.146	.145	.043	0.089
VA	.510	.093	.151	.044	0.076
IVA	.422	.073	.183	.033	0.077
IIIA	.380	.073	.155	.031	0.115
IIA	.418	.083	.126	.043	0.125
IA	.438	.083	.112	.039	0.140
IC		.063	.061	.039	0.141
IIC	.604	.051	.069	.023	0.143
IIIC	.833	.051	.036	.023	0.102
IIIB	.849	.083	.039	.021	0.087
IIB	.688	.072	.053	.016	0.085
IB		.062	.061	.006	0.080
XIV			.115	.059	0.075
XV			.098		
XVI			.062	.012	
XVII			.055	.059	
XIX			.077		
XXI			.077		
XXII			.057	.011	
XXIIA			.062	.016	
XXIA			.096	.018	
XVIA			.060	.031	
XVA			.148	.087	0.048
XVB			.206	.065	
XIVA			.180	.027	0.066

Table 8  
Old Board of Trade: Total Loads on Walls and Footings

From Report by Gen. Sooy Smith, dated May 4, 1893

Footings	Weight of Founda- tion (tons)	Weight of Super- structure (tons)	Total Load (tons)	Area of Footings (sq ft)	Soil Pressure (tons per sq ft)
I & IA	100.73	478.00	578.73	178.0	3.3
IB & IC	161.26	556.5	717.76	253.0	2.84
II & IIA	174.27	755.3	929.57	266.25	3.4
IIB & IIC	152.9	551.7	704.6	202.5	3.5
III & IIIA	152.34	606.15	758.49	196.0	3.8
IIIB & IIIC	251.56	1275.6	1527.16	518.0	2.95
IV & IVA	152.34	723.8	876.14	196.0	4.4
V & VA	152.34	606.15	758.49	196.0	3.8
VI & VIA	229.85	823.25	1053.1	348.0	3.0
VII & VIIA	149.06	684.8	833.86	267.0	3.12
VIII & VIIIA	113.27	653.8	767.07	206.0	3.72
IX	133.8	948.7	1082.38	277.5	3.9
IXA	117.5	722.5	840.0	236.3	3.6
X	149.03	769.9	918.93	246.0	3.73
XA	149.03	690.0	839.3	246.0	3.4
XI	128.29	603.9	732.19	183.3	4.0
XIA	128.8	634.7	763.5	183.3	4.1
XII	106.36	547.7	654.06	183.3	3.6
XIIA	111.1	563.17	674.27	183.3	3.7
XIII	131.76	576.0	707.76	205.30	3.45
XIIIA	131.36	531.6	662.95	205.3	3.23
XIV	81.17	559.0	640.17	154.0	4.15
XIVA	89.17	508.0	597.17	154.0	3.88
XV & XVA	103.65	605.5	709.15	199.5	3.6
XVII	96.47	411.5	507.97	260.0	1.95
XVIII	88.29	374.7	462.99	169.0	2.7
XIX	23.02	80.0	103.02	100.0	1.03
XXI & XXIA	58.42	319.87	378.29	121.0	3.13
XXII & XXIIA	67.34	280.5	347.84	121.0	2.9
XXIII & XXIIIA	82.14	666.44	748.54	289.0	2.6
E	184.13	1091.0	1275.13	414.0	3.08



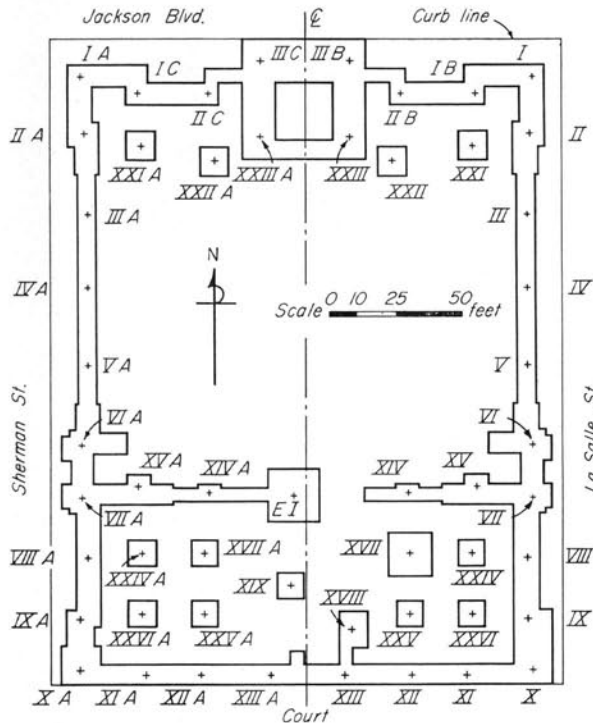


Fig. 36. Reconstructed Foundation Plan of Board of Trade

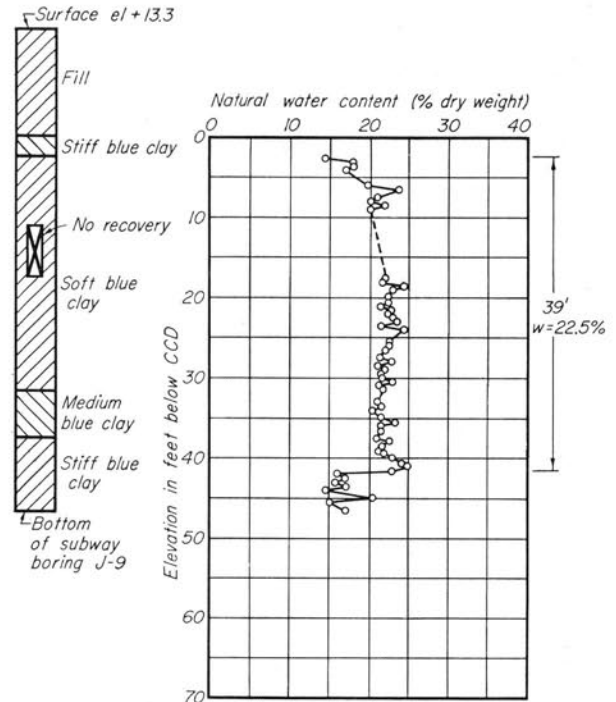


Fig. 37. Log of Boring near Board of Trade

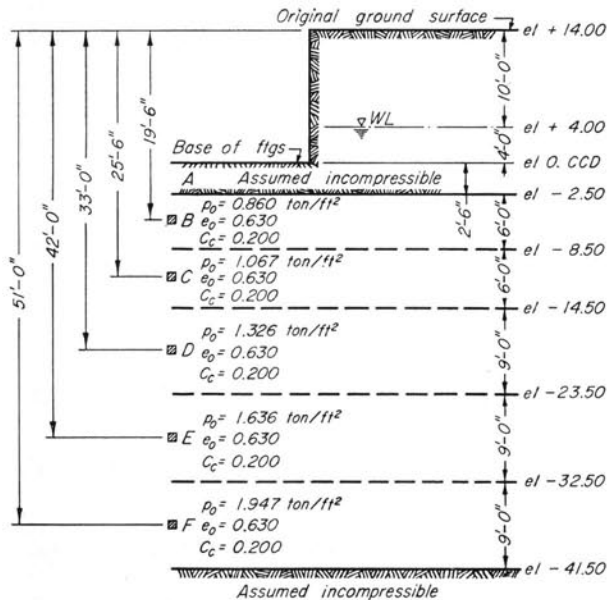


Fig. 38. Data for Computation of Settlement of Board of Trade

crease in settlement for different periods of time, and the increase in settlement for the period from 1889 to 1895 is unknown. The records after 1895 are continuous. A graphical method of estimating the magnitude of this gap in the time-settlement curve has been utilized. Three points on the east

Table 9  
Net Vertical Pressures under Old Board of Trade  
and Computed Settlements

Point	Pressure (tons/ft²) at depth indicated (ft)						Settlement (ft)
	6.25	12.50	18.75	25.0	31.25	37.5	
IIIB	2.384	1.618	1.052	0.829	0.671	0.428	1.306
XXIII	1.963	0.947	0.476	0.372	0.241	0.197	0.925
IIIB	1.937	1.055	0.667	0.522	0.398	0.315	1.056
IB	1.948	0.994	0.638	0.403	0.338	0.266	0.961
I	1.942	0.846	0.571	0.349	0.285	0.241	0.933
II	2.099	1.197	0.785	0.469	0.323	0.300	1.118
III	1.988	0.892	0.443	0.208	0.142	0.130	0.849
IV	1.802	0.987	0.474	0.237	0.099	0.074	0.828
V	1.944	0.906	0.441	0.210	0.091	0.085	0.832
VI	2.189	1.368	0.957	0.655	0.427	0.352	1.190
VII	2.129	1.345	0.921	0.716	0.499	0.463	1.200
VIII	1.989	1.092	0.720	0.584	0.490	0.456	1.096
IX	2.202	1.436	0.952	0.758	0.602	0.516	1.247
X	2.196	1.373	0.906	0.719	0.518	0.467	1.215
XI	2.039	1.073	0.708	0.537	0.443	0.411	1.086
XII	1.948	1.062	0.686	0.490	0.402	0.341	1.036
XIII	2.042	1.136	0.727	0.470	0.374	0.300	1.062
XIV	1.873	0.751	0.298	0.101	0.040	0.004	0.710
XV	1.930	0.977	0.554	0.450	0.289	0.276	0.962
EI	2.158	1.244	0.597	0.256	0.096	-0.020	0.917

wall (points III, IV, V) were selected for consideration because these points were far enough from the tower to be unaffected by its removal in 1895, and were also far from any deep excavation (see Fig. 30) that might have influenced their settlement. The known portion BCDE of the time-settlement curve for point IV, starting in 1895, was drawn as shown in Fig. 39. Point A in this figure represents the settlement from 1884 to 1889. The complete time-settlement curve must pass through the origin and point A and must be parallel to BCDE from 1895 on. Two different curves were

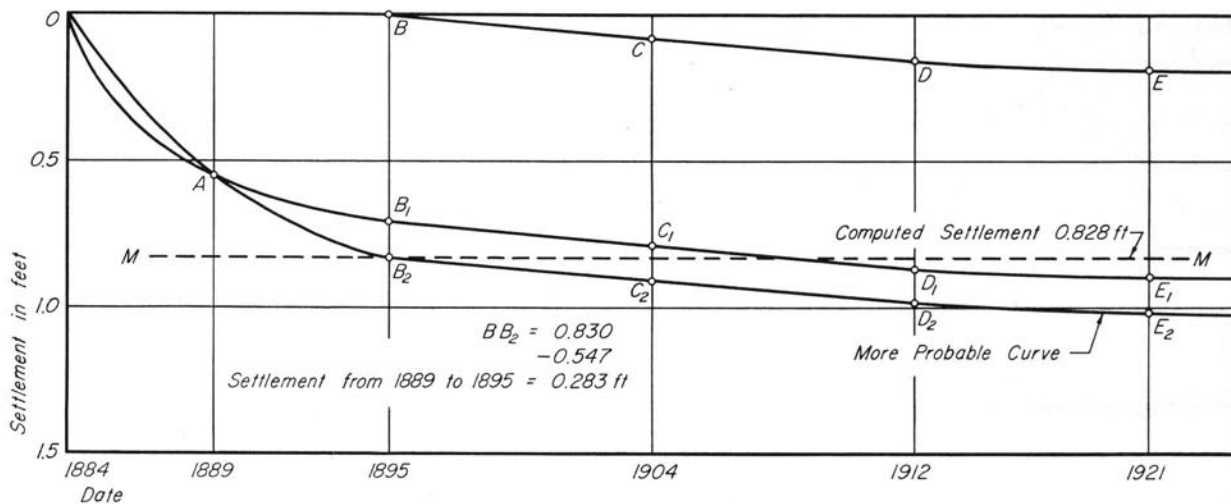


Fig. 39. Time-Settlement Curve of Point IV, Board of Trade

drawn that satisfied the given conditions. One curve represents the smallest reasonable amount of settlement for the period 1889 to 1895 ( $AB_1$ ) and the other the greatest ( $AB_2$ ). The computed settlement for point IV is 0.828 ft. Since the actual settlements are usually greater than the computed ones, the curve  $OAB_2C_2D_2E_2$  was assumed to be the more probable time-settlement curve for point IV. It is shown drawn to a semi-logarithmic scale in Fig. 40. The same procedure was followed in determining the time-settlement curve for points III and V. Settlements at 100 percent primary consolidation were determined by the graphical procedure for all three points, and the ratios of these to the settlements for the period 1884 to 1889 were taken. The average of these three ratios was found to be 1.7 and was assumed to be constant for all the other points in the building. Settlements from 1884 to 1889 for all points given in Table 7 and Fig. 35

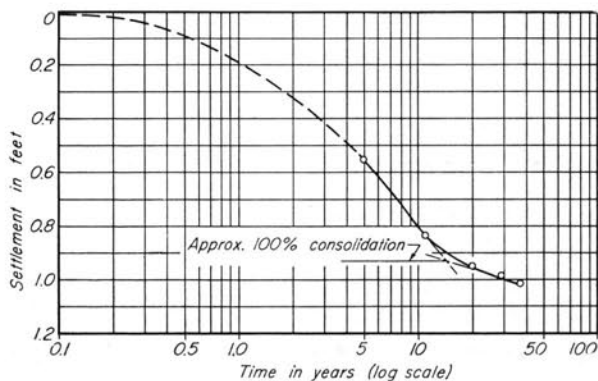


Fig. 40. Semi-Logarithmic Plot of Time-Settlement Relation for Point IV, Board of Trade

were multiplied by 1.7 and considered to be the observed settlements. This is a rather rough approximation, but on account of the lack of complete settlement records it was used to permit comparison of actual with computed settlements.

## 22. Discussion of Computed and Observed Settlements

Figure 41 shows both the computed and the actual settlements along three walls of the building.

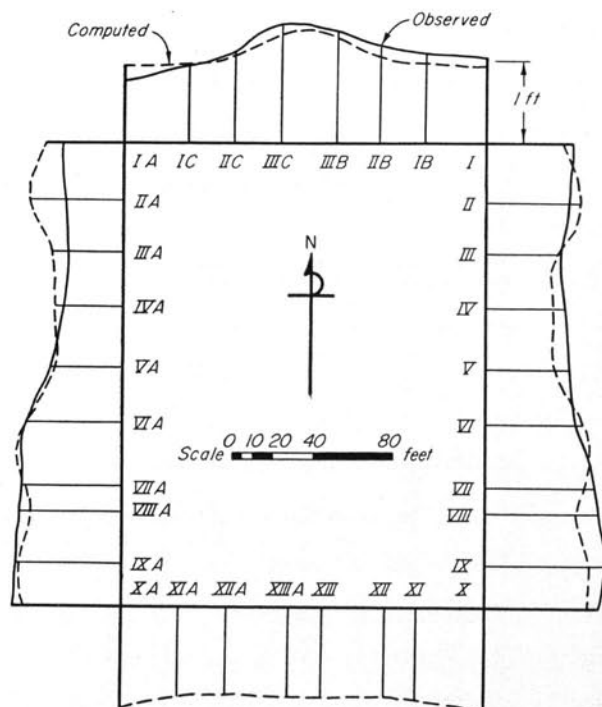


Fig. 41. Computed and Observed Settlements of Board of Trade

Figure 42 shows settlement profiles and stress patterns along the east and north walls.

In view of all the assumptions made in determining the computed as well as the actual settlements, the results agree surprisingly well. Better agreement between the actual and computed settlements could hardly be expected. However, the

curve of observed settlements shows less differential settlement along the side walls than the curve of computed settlements, but more along the north wall. Furthermore the actual settlements are not exactly symmetrical, although the deviation from symmetry is small.

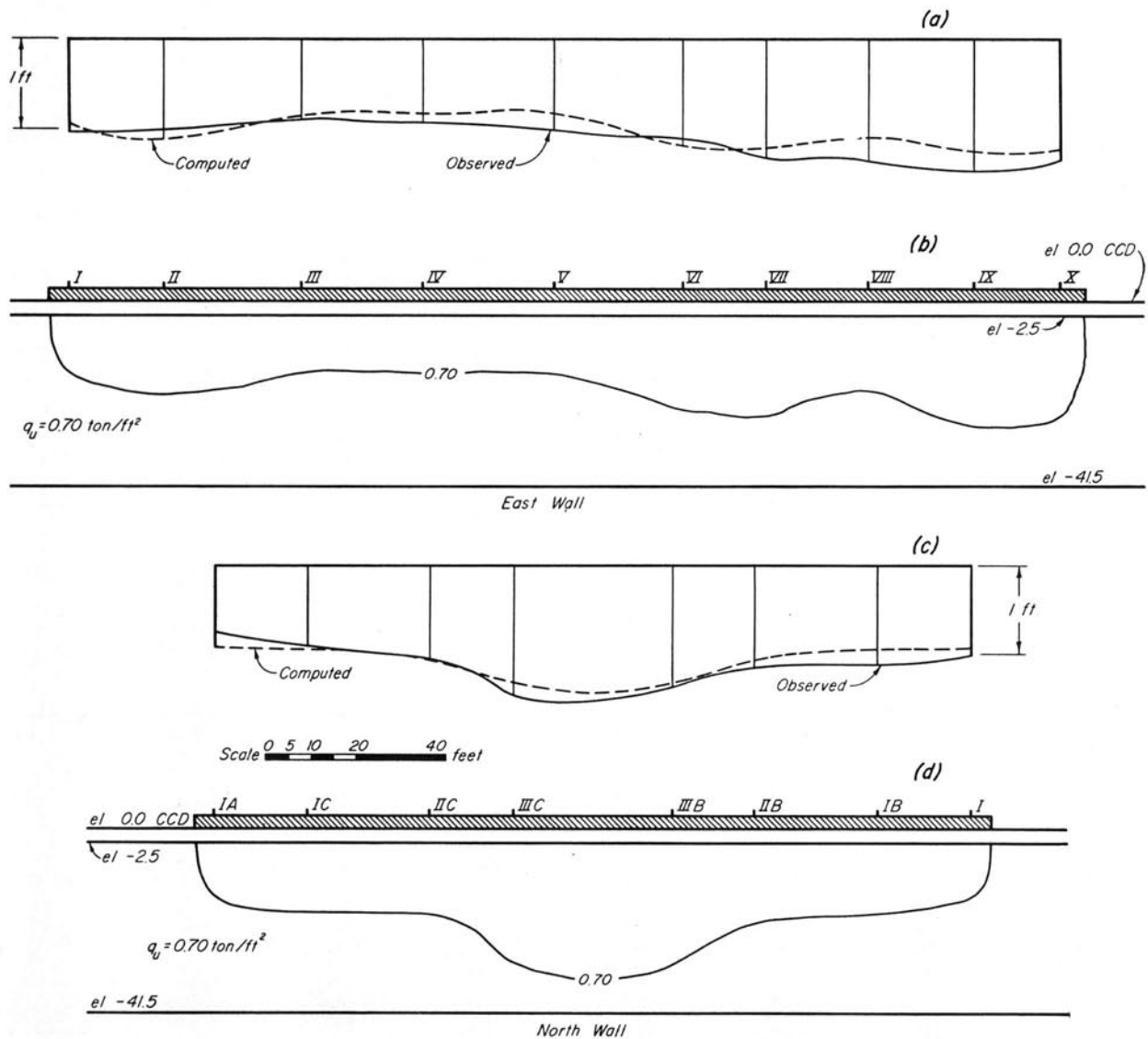


Fig. 42. Comparison for Old Board of Trade of (a and c) Computed and Observed Settlements; (b and d) Net Vertical Pressure in Subsoil and Unconfined Compressive Strength (tons per sq ft)

## VI. APARTMENT BUILDING

### 23. Information Available

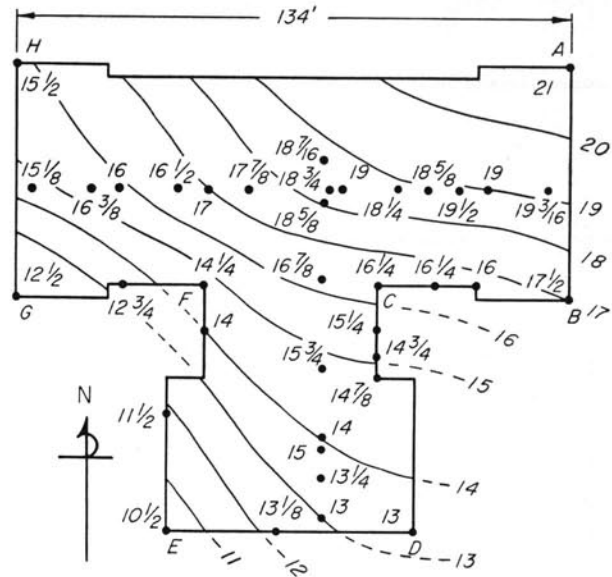
This building is a 19-story skeleton structure with a reinforced-concrete frame, founded on a heavily reinforced mat 3 ft thick. It is T-shaped in plan, as shown in Fig. 43. The site was excavated for a depth of 14 ft to El. + 1.0. The mat was then placed and the frame constructed. One wing of the mat was reserved for use as the boiler room of the structure. Beneath this area, the unit soil pressure due to dead load is 4000 lb per sq ft. The remainder of the mat was backfilled to the level of the first floor; beneath this area the soil pressure is 4820 lb per sq ft. The locations of the loaded areas are shown in Fig. 47 (page 43).

The structure was erected in 1924. There is no evidence that it was set above grade to allow for settlement. In 1934, levels were taken along the corridors of the 4th and 12th floors and along the base course of stone on the exterior. Settlements determined on the assumption that these elements of the structure were originally constructed at their theoretical evaluations are shown in Fig. 43. In 1934, points were established around the periphery of the building, and level readings taken at intervals until 1940. The results of these surveys are also shown in Fig. 43. The structure is located between older buildings and has not been affected by adjacent excavation.

Soil conditions are indicated by a boring made by R. C. Smith at the site, and by a Subway boring some 300 ft west. The boring data are given in Fig. 44.

### 24. Assumptions for Computation

The seat of settlement was assumed to lie between El. -3.0 and El. -39.0 as shown in Fig. 44. The compressible material was divided into two layers of different water content. The upper layer with a thickness of 15 ft was assumed to have a water content of 28.0 percent and was divided into three layers of 5 ft each. The lower layer with a thickness of 21 ft was assumed to have a water content of 21.0 percent and was divided into two layers of 10.5 ft each. The layers and the corresponding constants for computation are shown in



SETTLEMENT RECORD IN INCHES

Reading date	A	B	C	Point D	E	F	G
5-27-34	Initial	readings	established				
6-30-34	0	0	0	1/16	1/16	0	0
7-30-34	1/8	1/8	1/16	1/16	1/16	0	0
9-1-34	1/8	1/8	1/16	1/16	1/16	1/16	1/16
10-1-34	1/8	1/8	1/8	1/8	1/8	1/8	1/8
12-5-34	3/16	1/8	1/4	1/8	1/8	1/4	3/16
4-20-35	1/4	5/16	3/8	5/16	5/16	1/4	1/4
9-2-37	1	15/16		1		1	1
8-30-38	1 1/4	1 1/8	1 1/4	1 1/4	1 1/4	1 1/4	1 1/4
5-9-40	1 3/8	1 1/4	1 5/8	1 7/16	1 1/2	1 9/16	1 1/2

TYPICAL TIME - SETTLEMENT CURVE

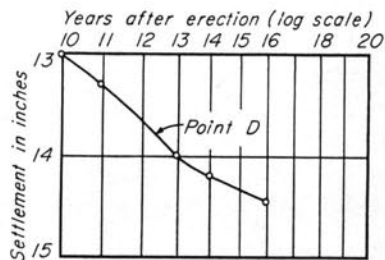


Fig. 43. Settlement of Apartment Building in 1934





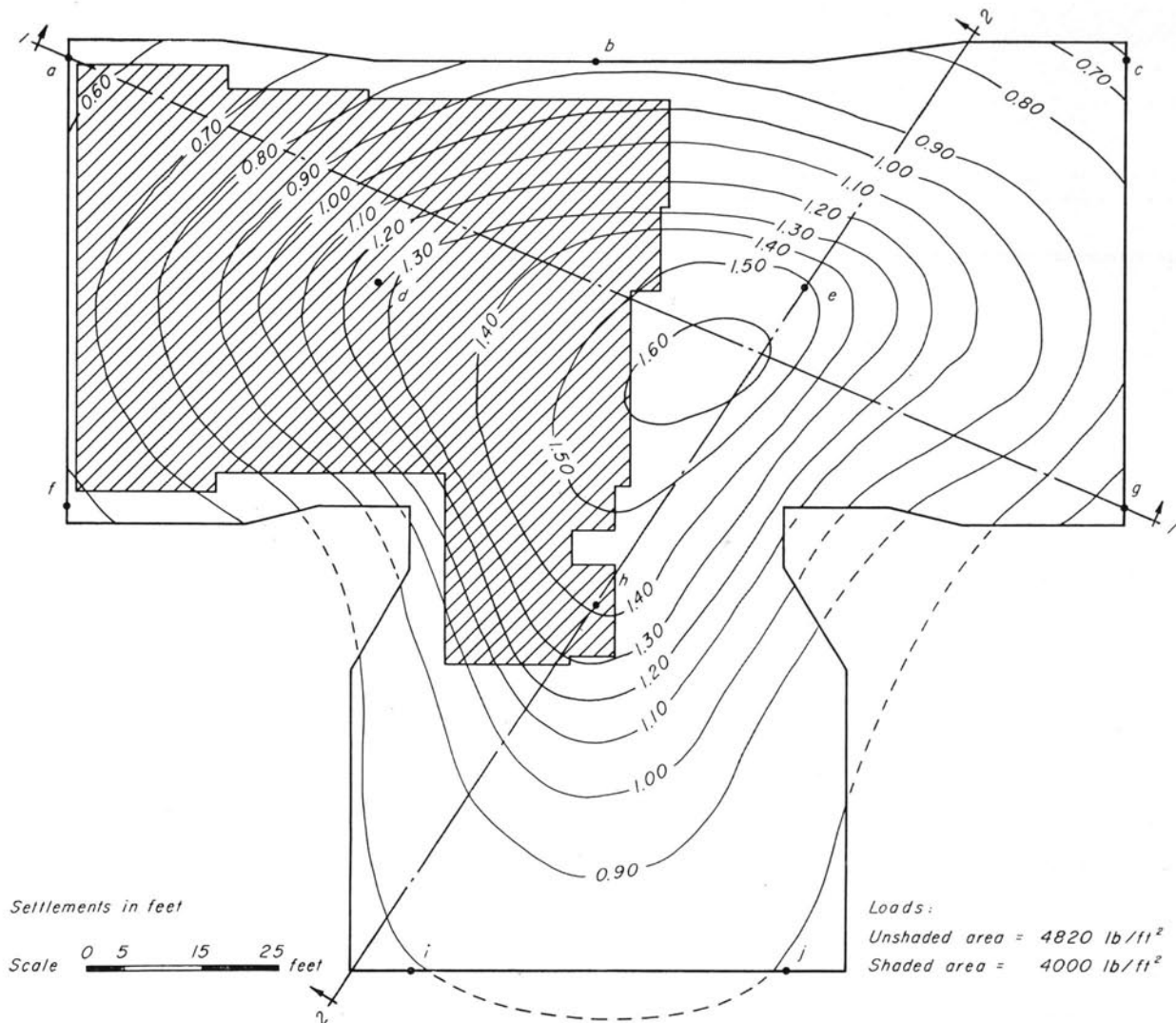


Fig. 47. Computed Primary Settlement and Assumed Contact Pressure for Apartment Building on Assumption of Complete Flexibility

tion  $K$  does not give more accurate results than the results obtained from the assumption of a perfectly flexible mat. All the stresses and settlements computed for each case are given in Tables 10 and 11.

## 25. Observed Settlements

There are no complete time-settlement records for any point in the building. However, Fig. 43 shows the results of settlement observations made during the period 1934 to 1940. The rate of settle-

Table 10  
Net Vertical Pressures under Apartment Building and  
Computed Settlements  
(Structure Assumed Flexible)

Point	Pressure (ton/ft <sup>2</sup> ) at depth indicated (ft)					Settlement (ft)
	6.25	12.5	18.75	25.0	37.5	
a	0.539	0.426	0.387	0.357	0.335	0.576
b	0.771	0.717	0.696	0.661	0.661	0.887
c	0.598	0.507	0.472	0.443	0.413	0.683
d	1.214	1.187	1.167	1.103	0.967	1.284
e	1.615	1.566	1.500	1.384	1.158	1.524
f	0.549	0.439	0.397	0.380	0.376	0.592
g	0.598	0.508	0.474	0.461	0.451	0.673
h	1.321	1.384	1.364	1.302	1.135	1.417
i	0.764	0.666	0.592	0.548	0.475	0.786
j	0.764	0.666	0.592	0.548	0.480	0.791

Table 11  
Net Vertical Pressures under Apartment Building and  
Computed Settlements  
(Constant Coefficient of Subgrade Reaction)

Point	Pressure (ton/ft <sup>2</sup> ) at depth indicated (ft)				Settlement (ft)
	0	12.5	23.5	44.0	
a	0.352	0.349	0.344	0.318	0.478
b	0.803	0.784	0.793	0.701	0.959
c	0.452	0.445	0.431	0.385	0.584
d	1.405	1.393	1.240	1.018	1.399
e	1.680	1.621	1.431	1.121	1.542
f	0.314	0.315	0.333	0.348	0.458
g	0.420	0.416	0.425	0.418	0.562
h	1.405	1.388	1.277	1.039	1.412
i	0.295	0.297	0.295	0.316	0.424
j	0.333	0.332	0.326	0.334	0.466



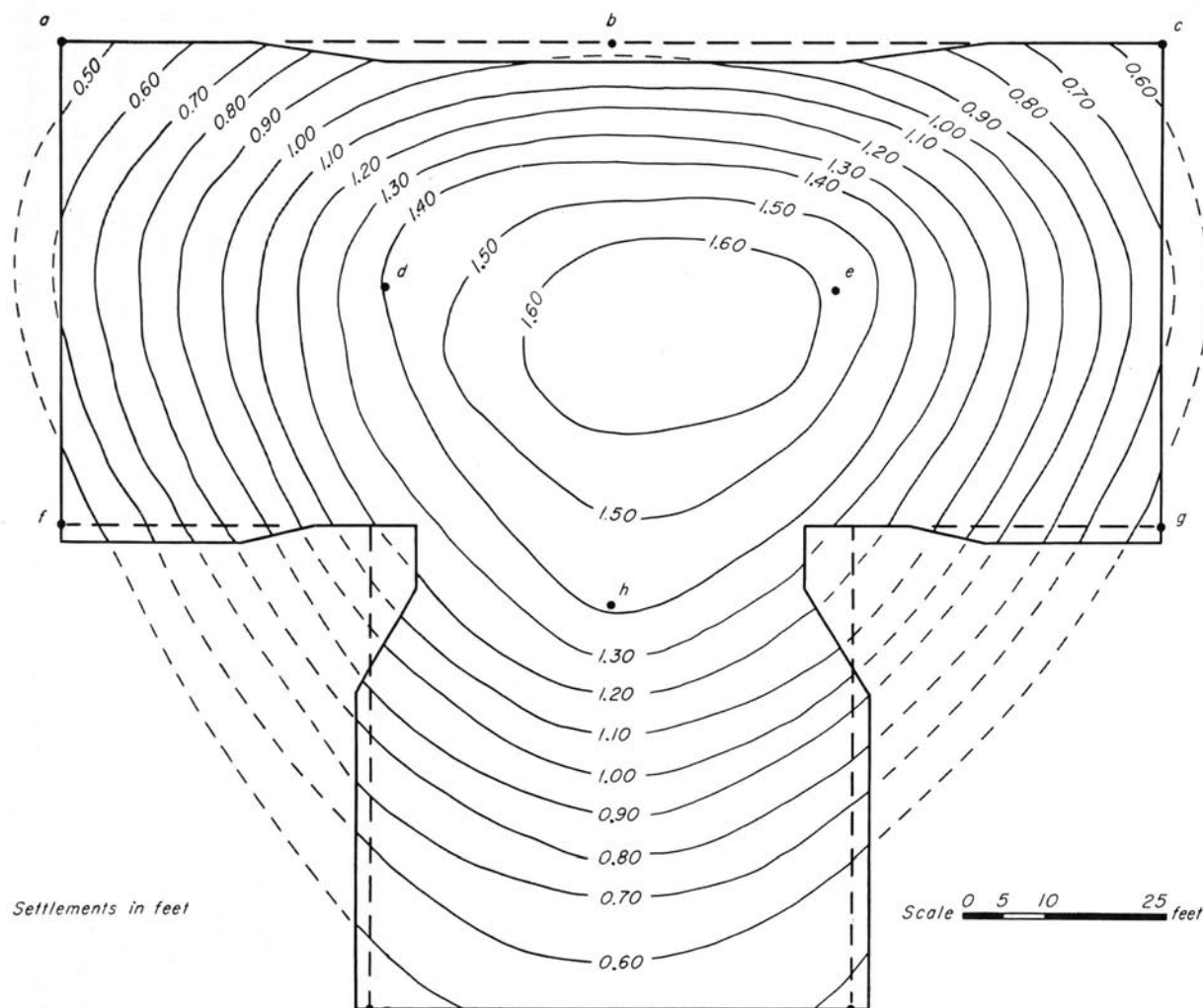


Fig. 48. Computed Primary Settlement of Apartment Building for Contact Pressure Corresponding to Constant Modulus of Subgrade Reaction

ment during this period is generally very small. This suggests that the primary consolidation in the soil may have ended and the secondary time effect started. Therefore the settlements corresponding to 100 percent consolidation are probably not greater than those observed in 1934, and the 1934 settlements have been taken as the observed settlements to be compared with the computed ones.

## 26. Discussion of Computed and Observed Settlements

The contours of observed settlement, Fig. 43, are roughly straight, parallel, equally-spaced lines running in the northwest-southeast direction. They leave no doubt that the mat remained very nearly plane during the settlement process, although it tilted. On the other hand, the contours of computed settlement, Figs. 47 and 48, indicate a bowl-shaped

settlement regardless of which assumption was used to compute the stresses.

The similarities and differences between the observed and computed settlements can best be studied with reference to the settlement profiles, Fig. 49. The location of the profiles is shown in the figure. It is apparent that the average observed settlement exceeds the average computed settlement by 10 to 15 percent. Furthermore, in spite of the bowl-shaped appearance of the calculated settlement curves, they clearly indicate the non-symmetrical nature of the settlement.

Figure 50 shows the same settlement profiles (Section 2-2) as Fig. 49, and in addition shows the relation of the computed stresses to the unconfined compressive strength of the soil. It is apparent that the entire compressible clay deposit is highly stressed beneath the building.

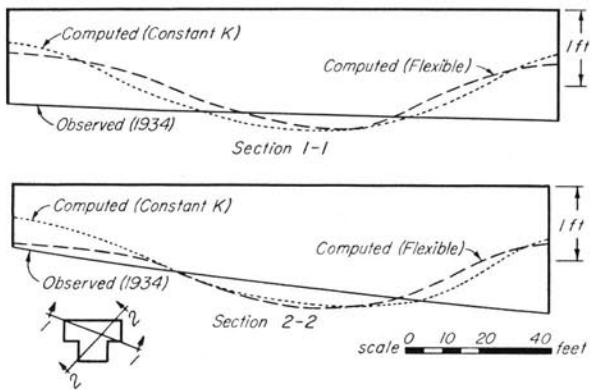


Fig. 49. Computed and Observed Settlements for Apartment Building

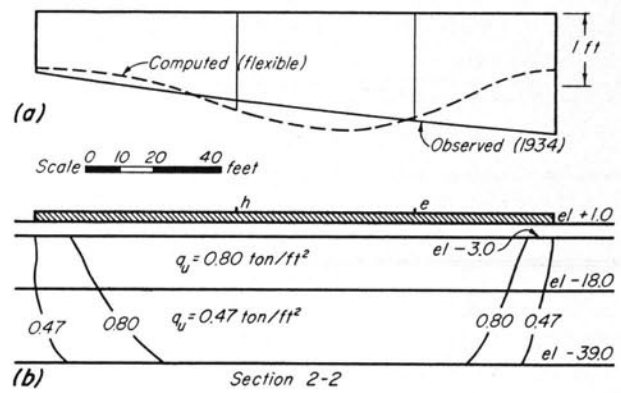


Fig. 50. Comparison for Apartment Building of (a) Computed and Observed Settlements; (b) Net Vertical Pressure in Subsoil and Unconfined Compressive Strength (tons per sq ft)

## VII. POLK STREET STATION

### 27. Information Available

The Polk Street Station is located on the south side of Polk St at the foot of Dearborn St. Originally, it consisted of a tower about 22 ft square and 170 ft high flanked by 2-story sections with gable roofs, and with 3-story sections at the ends of the Polk St front (see Fig. 51).

The structure was built in 1884, and was then known as the Chicago and Western Indiana Station. The upper portion of the tower was destroyed by fire at an unknown date, probably in the early 1900's, and was not rebuilt above a height of about 100 ft. Furthermore, during the same general period, the gable roof of the 2-story section was removed and a third story added.

The foundation plan of the structure, together with the other architectural drawings, was made available by the chief engineer of the C. & W.I.R.R. The structure was founded on masonry footings, placed as shown in Fig. 52. The base of the tower footing was at El. 0.0. That of the other footings was at about El. +2.0. The basement floor was located at about El. +4.0. There were no sub-side-walk vaulted spaces.

Settlement levels were taken by the C. & W.I. R.R. on April 3, 1899, along the Polk St front to determine the settlement of the curb and sidewalk. These levels were referred to curb grade, El. +14.0. They indicated that the settlement was concentrated near the tower, and that near the east end of the eastern 2-story portion the sidewalk adjacent to the building was 9 in. above curb grade. Since the sidewalk here was 24 ft wide, its slope corresponded to  $\frac{3}{8}$  in. per ft, a commonly used value. This suggests that practically no settlement had occurred in this location. At the doorway in this locality is a 6-in. step up to floor level. Hence, inside grade was probably originally at El.  $14.00 + 0.75 + 0.50 = \text{El. } 15.25$ .

In 1943, the Joint Committee surveyed the structure and found the elevation of this doorway to be 15.20. Hence, it is safe to assume that the doorway in 1943 was within about  $\frac{5}{8}$  in. of its original elevation. With El. 15.20 as a reference, differential levels were run along lines originally horizontal, such as the stone base courses and window ledges. From these data, the settlement profile shown in Fig. 52 was constructed. It repre-

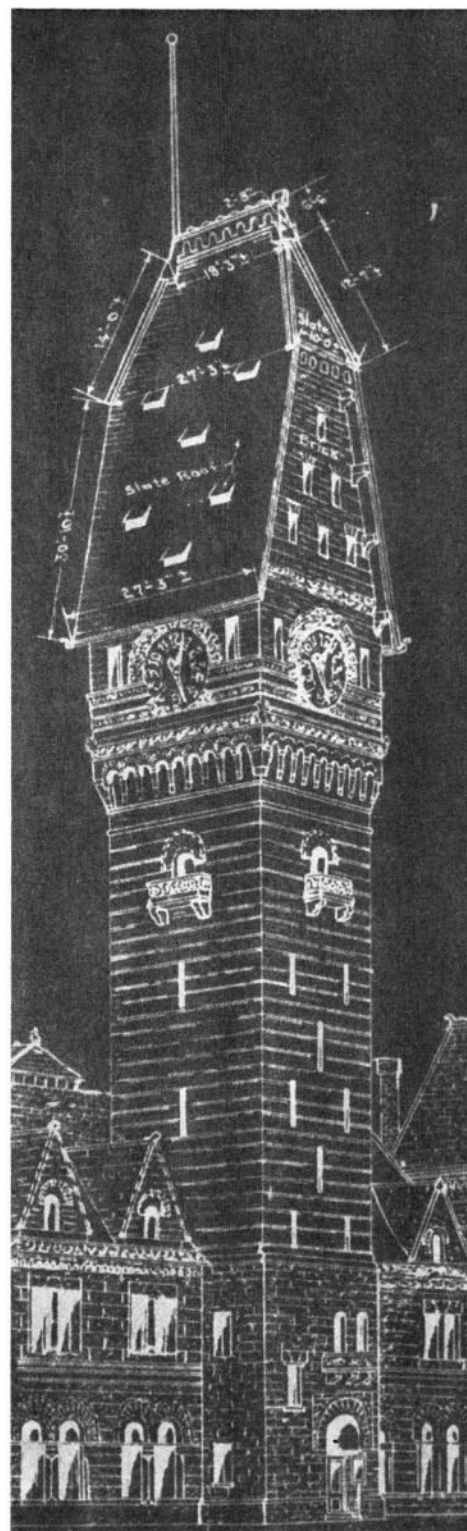


Fig. 51. Polk Street Station with Original Tower

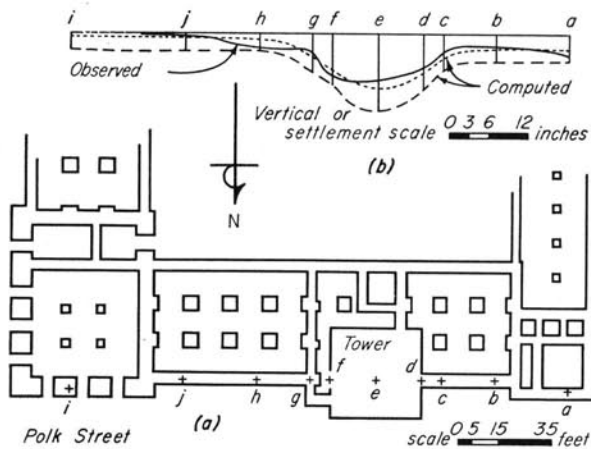


Fig. 52. Polk Street Station (a) Foundation Plan; (b) Settlements along North Wall in 1943

sents the settlement of the Polk St front of the building from 1884 to 1943. Furthermore, the profile is practically identical with that obtained in 1899 by observing sidewalk elevations adjacent to the building. It appears, therefore, that the settlement between 1899 and 1943 was not more than  $\frac{5}{8}$  in. The partial removal of the tower shortly after 1899 may have arrested the tendency of this portion of the building to settle. Other parts of the building were always very lightly loaded.

No reliable boring has been made at the site of the station. The nearest Subway boring is at the corner of Polk and State Sts, about 500 ft east of the building.

## 28. Assumptions for Computation

There was no information available concerning the soil pressure at the base of the footings. Hence a careful estimate of dead load for one portion (2-story section) of the building was made from the available architectural drawings. The soil pressure at the base of the footings was found to be 3300 lb per sq ft. To simplify the stress computations, all the footings were assumed to be founded at El. +0.70.

The Subway boring, Fig. 53, was assumed to indicate the soil conditions. Since the boring is over a block away, this assumption may not be fully justified. The seat of settlement was assumed to lie between El. -2.5 and El. -40.5. The water content for the first 9.0 ft was assumed to be 36.0 percent and this rather soft layer was divided into two 4.5-ft layers. The remaining 29.0 ft of the seat of settlement were assumed to have a water content of 23.0 percent and were divided into three layers 9.0, 10.0 and 10.0 ft thick as shown in Fig. 54.

Computed stresses as well as settlements under points indicated on the footing plan as *a*, *b*, . . . *i*, in Fig. 52 are shown in Table 12.

## 29. Observed Settlements

The time-settlement relation is not known for any point in the building. The total settlements obtained from the survey made in 1943 by the Joint Committee were assumed to be the observed

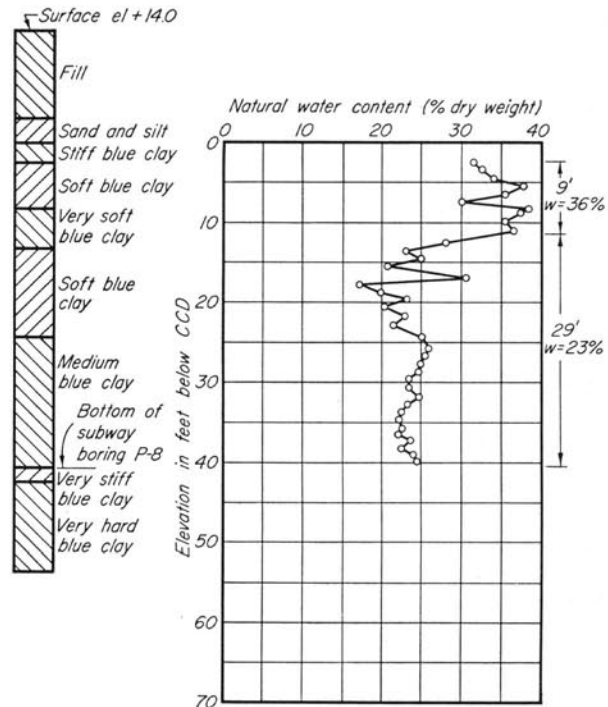


Fig. 53. Log of Boring One Block from Polk Street Station

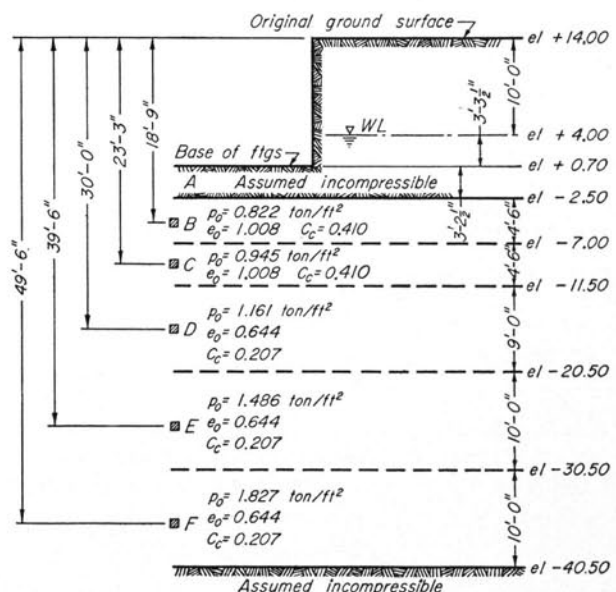


Fig. 54. Data for Computation of Settlement of Polk Street Station

**Table 12**  
**Net Vertical Pressures under Polk Street Station**  
**and Computed Settlements**

Point	Pressure (ton/ft <sup>2</sup> ) at depth indicated (ft)						Settle- ment (ft)
	6.25	12.5	18.75	25.0	37.5	50.0	
a	0.301	0.140	0.093	0.081	0.052	0.039	0.303
b	0.307	0.137	0.072	0.085	0.069	0.069	0.316
c	0.295	0.190	0.176	0.179	0.134	0.108	0.404
d	0.558	0.412	0.345	0.285	0.185	0.136	0.661
e	0.885	0.763	0.576	0.454	0.261	0.170	0.965
f	0.396	0.446	0.368	0.310	0.199	0.135	0.616
g	0.417	0.275	0.225	0.194	0.140	0.103	0.498
h	0.241	0.053	-0.004	0.012	0.011	0.028	0.200
i	0.298	0.045	-0.032	-0.036	-0.030	-0.024	0.211
j	0.256	0.084	0.013	0.002	-0.019	-0.018	0.214

settlements. This appears justified by the conclusion that the settlement increased from 1899 to 1943 by only about  $\frac{5}{8}$  in.

### 30. Discussion of Computed and Observed Settlements

Figure 55 shows a settlement profile indicating both computed (dash line) and observed (full line) settlements. This profile indicates that even though the observed settlement includes the secondary time effect, the computed settlements are greater than the observed ones along the entire front wall. A possible reason for this exceptional result is that the soil information from the Subway boring, shown in Fig. 53, may not accurately represent the conditions beneath the building.

For example, according to the level surveys the east wing of the building did not settle at all whereas the west wing settled as much as 2.0 in. However, the pressure variation shown in Fig. 55 leaves no doubt that there should have been an appreciable settlement at the east end, even if the

compressible clay below the tough crust were on the average considerably stiffer than the boring indicates. On the other hand, if it is assumed that the incompressible tough clay layer (layer A, Fig. 54) extends as deep as El. -7.0, the computed settlement at the east end reduces to zero. This assumption leads to the computed settlement profile indicated by the dotted line in Fig. 52. This profile agrees with the observed settlements much better than the computed values based on the previous assumptions. In the absence of conclusive evidence, it can be considered probable that the stiff clay crust is unusually thick beneath this building.

It should also be remembered that the load of the tower, which is the main cause of differential settlement, was reduced when the upper portion of the tower was destroyed by fire. This probably caused the settlement of the tower to cease. Since most of the primary settlement had occurred by the time of the fire, the computation was based on the assumption that the original load of the tower acted throughout the duration of the primary settlement. However, if the primary settlement had not been completed at the time of the fire, the computed settlement of the tower would be too great.

In view of the uncertainties in soil conditions and loads, the computed and observed settlements are in reasonable agreement.

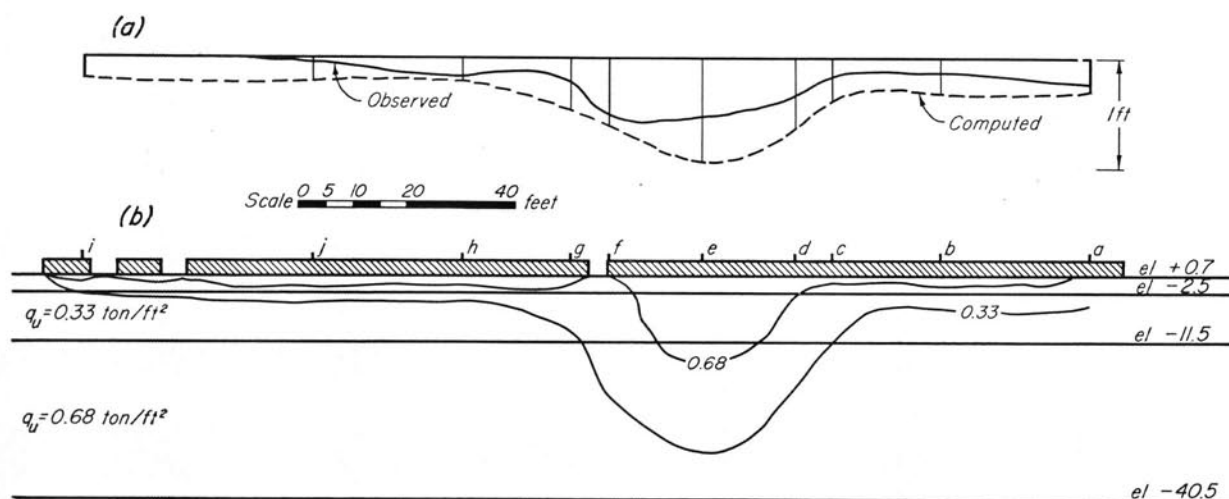


Fig. 55. Comparison for Polk Street Station of (a) Computed and Observed Settlements; (b) Net Vertical Pressure in Subsoil and Unconfined Compressive Strength (tons per sq ft)



## VIII. JUDSON WAREHOUSE

### 31. Information Available

The Judson Warehouse is located on the west side of State St, south of 11th St. It is a 2-story structure, 60 by 326 ft in plan, designed as a warehouse and built in 1929 or 1930. Three rows of columns run from north to south; the middle row divides the building into two bays. The rear bay is backfilled to bring the floor elevation to +17.2, whereas the front bay is paved at street level, El. +13.2, to allow trucks to unload under the protection of the second story.

The structure rests on concrete footings founded at Els. +9.2, +7.7, and +5.7 at the rear, middle,

present dead loads can be calculated. The structure has been used principally for transfer of goods from truck to train, and the permanent live load is believed to have been relatively small.

The design drawings also show the theoretical elevations of all elements of the structure, including the rolled steel floor beams. In 1941, the Joint Committee determined the elevations of the exposed bottom flanges of these beams at the middle row of columns throughout the length of the building, and at the front and rear columns at two cross-sections. These levels were run after construction of the

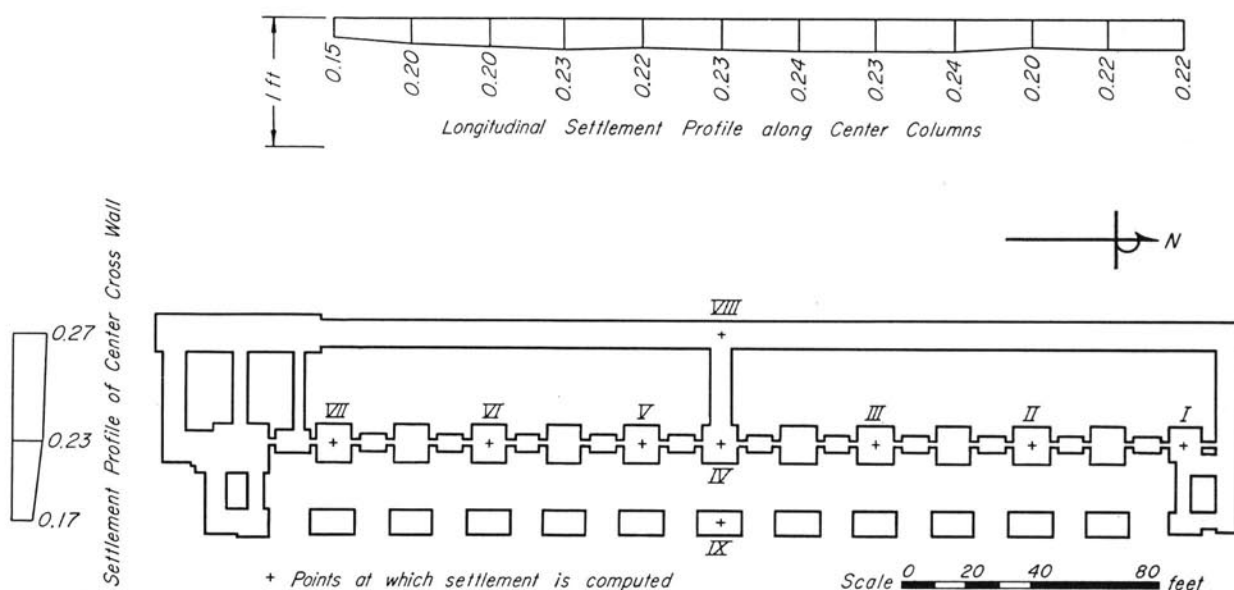


Fig. 56. Foundation Plan of Judson Warehouse

and front lines respectively, except in the southernmost 39 ft of the building where the rear bay is occupied by a basement (see footing plan in Fig. 56) with its floor at El. +4.2. The footings in the middle and rear lines are here founded at El. -0.3.

The warehouse was designed for an allowable soil pressure of 3000 lb per sq ft, but the present two stories are only a portion of the structure ultimately planned. Design drawings of the A.T. and S.F. Ry. System are available from which the

State Street subway. During subway construction, however, accurate levels were taken by the City to determine the amount of settlement due to the excavation. The elevations of the Joint Committee's survey were increased by the amount of settlement due to subway construction in order to obtain the elevation of the reference points just prior to subway construction in 1940. The corresponding settlements are shown in Fig. 56. They are likely to be in error by not more than  $\frac{1}{2}$  in.; the principal

source of error is probably inaccuracy in setting the steel floor beams.

The subsoil at the site contains some of the softest clay in the Chicago region. The closest Subway boring is located in State St 314 ft south of 11th St, directly opposite the warehouse.

### 32. Assumptions for Computation

Since the building is not yet complete according to the original plans, the maximum soil pressure used for design is not exerted by the present building. Hence, an estimate of the soil pressure was made from the available drawings. This pressure was found to be 1400 lb per sq ft at the base of the footings.

The difference in elevation of the bases of the footings was disregarded in the stress computation, and El. +7.7 was assumed to be the elevation of the base of all footings in the north portion of the building where no basement exists. The elevation of the footings in the south portion of the building, where there is a basement, was taken as -0.3. The difference in elevation of the footings in the two portions of the building was taken in consideration in computing the stresses under the building.

Seven points were selected along the middle line of columns in the north-south direction (see footing

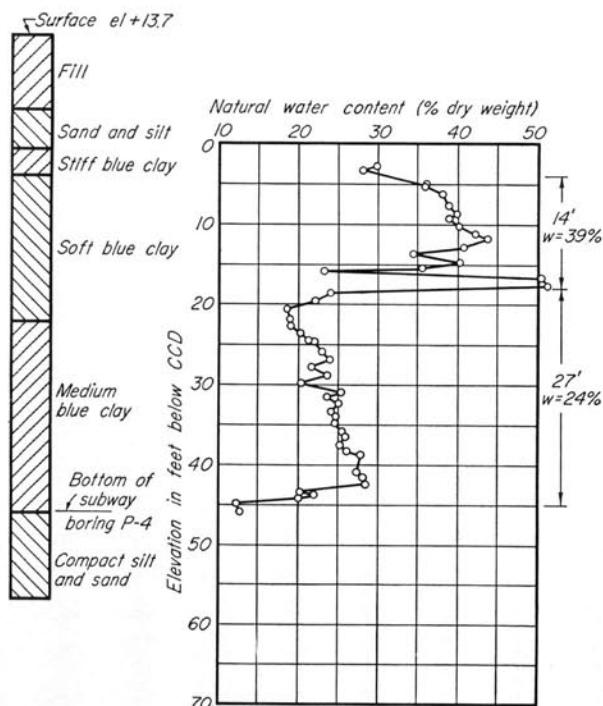


Fig. 57. Log of Boring near Judson Warehouse

plan in Fig. 56) and two more points along the east-west wall in the middle of the building. These points are numbered I to IX (see Fig. 56).

The soil conditions under the building are shown in Fig. 57. The original ground surface was assumed to be at El. +14.0 and the seat of settlement was taken as the layers between El. -4.0 and El. -45.0. The upper 14 ft of the seat of settlement were assumed to have a water content of 39 percent and were divided into two 7-ft layers. The remaining 27 ft were assumed to have a water content of 24.0 percent and were divided into three 9-ft layers. Figure 58 indicates all the layers with the constants needed for computation of the settlement.

With these assumptions, stresses and settlements were computed under each of the nine points mentioned above. The results are presented in Table 13.

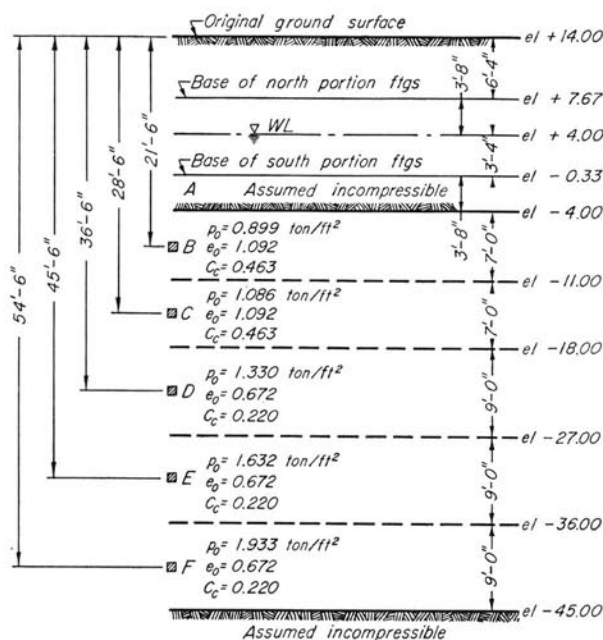


Fig. 58. Data for Computation of Settlement of Judson Warehouse

Table 13  
Net Vertical Pressures under Judson Warehouse  
and Computed Settlements

Point	Pressure (ton/ft <sup>2</sup> ) at elevation indicated (ft)					Settlement (ft)
	-4.75	-11.00	-17.25	-29.75	-42.25	
I	0.157	0.133	0.109	0.082	0.064	0.244
II	0.143	0.105	0.084	0.077	0.067	0.215
III	0.143	0.105	0.084	0.078	0.068	0.215
IV	0.163	0.123	0.103	0.088	0.073	0.247
V	0.145	0.109	0.088	0.082	0.071	0.225
VI	0.143	0.105	0.085	0.073	0.062	0.210
VII	0.117	0.043	0.001	-0.004	-0.004	0.073
VIII	0.148	0.120	0.099	0.076	0.064	0.245
IX	0.113	0.077	0.066	0.060	0.056	0.165

### 33. Observed Settlements

There was no time-settlement record available for any point in the building. Hence, the settlement profile obtained from the survey made during subway construction in 1942, and presented with the footing plan in Fig. 56, was used for comparison with the computed settlements. Inasmuch as the observed settlements correspond to a nine-year period after construction, they probably correspond to somewhat less than 100 percent consolidation.

### 34. Discussion of Computed and Observed Settlements

Figure 59 shows the settlement profile and the variation in pressure under the cross-wall and under the north-south center line. These settlement curves, computed and observed, agree almost perfectly except under point VII. At this point both the computed and observed settlements have their least values along the entire north-south profile. This is reasonable, because the excavation stresses are greater here than under any other point.

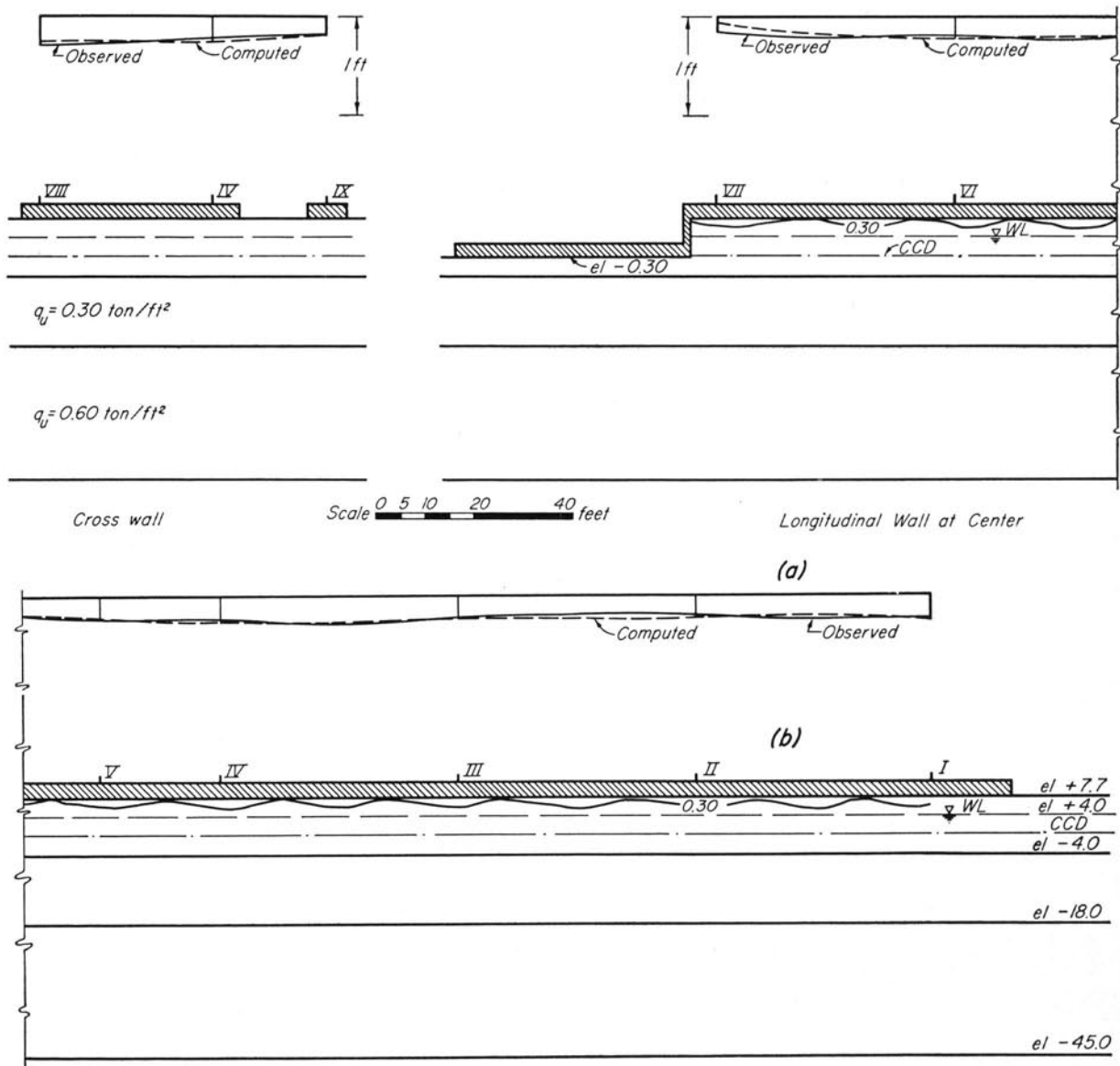


Fig. 59. Comparison for Judson Warehouse of (a) Computed and Observed Settlements; (b) Net Vertical Pressure in Subsoil and Unconfined Compressive Strength (tons per sq ft)

## IX. RELIABILITY OF COMPUTED SETTLEMENTS

### 35. Simplified Procedure for Computing Settlements

A comparison between the observed and computed settlements can be made in a general way by studying the various drawings in the text showing the settlement contours or profiles. Some conclusions based on such studies have already been drawn in connection with the individual buildings.

Further information can be obtained by a more quantitative comparison between the computed and observed settlements of all the buildings considered as a group. To facilitate a comparison of this type it is advantageous to express the computed settlements in an approximate but simple manner. In the following paragraphs, such an approximate procedure for estimating settlements is developed. The simplified procedure involves no basic assumptions different from those used in the detailed settlement computations. Simplifying assumptions, however, are made concerning the values of  $e_0$ ,  $C_c$ ,  $p_0$  and  $H$ , Eqs. 1 and 2.

### 36. Development of Limiting Equations

The equation for settlement is

$$S = \frac{\Delta e}{1 + e_0} H \quad (2)$$

By substituting

$$\Delta e = C_c \log \frac{p_0 + \Delta p}{p_0} \quad (1)$$

we obtain

$$S = \frac{C_c \log \left(1 + \frac{\Delta p}{p_0}\right)}{1 + e_0} H \quad (4)$$

The quantities to be evaluated approximately in Eq. 4 are

- $e_0$  Initial void ratio of the consolidating clay
- $C_c$  Slope of virgin compression curve of the consolidating clay
- $p_0$  Initial pressure on the clay prior to the application of the building load
- $H$  Thickness of the consolidating layer.

Since the clay is fully saturated, we may write  $e_0 = 2.8 w$  where 2.8 is assumed to be the specific gravity of the solid matter and  $w$  is the water content of the clay expressed as a ratio of the weight

of water to the dry weight of the soil. Furthermore, the equation of the straight line in Fig. 2 is

$$C_c = 1.766 w^2 + 0.593 w - 0.0135 \quad (5)$$

For clays in the Chicago business district, the quantity  $w$  can be replaced by appropriate approximate values. Two limiting values of water content, one for very soft and one for medium soft clay, are assumed within the general range of variation. These values are

$w = 36.0$  percent for very soft clay, corresponding to  $e_0 = 1.008$

and

$w = 22.0$  percent for medium soft clay, corresponding to  $e_0 = 0.616$ .

Substituting these quantities in Eq. 5, we obtain

$C_c = 0.4285$  for very soft clay and

$C_c = 0.2025$  for medium soft clay.

By substituting these values as well as the values of the corresponding values of  $e_0$  in Eq. 4, two limiting equations for settlement are obtained:

$$S_{vs} = 0.213 H \log \left(1 + \frac{\Delta p}{p_0}\right) \quad (6)$$

for very soft clays and

$$S_{ms} = 0.125 H \log \left(1 + \frac{\Delta p}{p_0}\right) \quad (7)$$

for medium soft clays.

The value of  $H$  can be approximated by choosing various thicknesses for the compressible part of the deposit and computing settlements by means of Eqs. 6 and 7 until the thickness is found that gives settlements in best agreement with those computed by the more elaborate general procedure. For any assumed value of  $H$ , the value of  $p_0$  at the middle of the layer can be computed. All the other general assumptions made in Article 6 are retained, with the addition that the stiff desiccated crust is always considered to lie between El. 0.0 and El. -3.0. The expressions for  $p_0$  (tons per sq ft) in terms of  $H$  (ft) become

$$(p_0)_{vs} = 0.773 + 0.01395 H \quad (8)$$

$$(p_0)_{ms} = 0.773 + 0.01740 H \quad (9)$$

The difference is due to the variation of the unit weight of the clay with water content.

By trial and error it was found that the most satisfactory expressions for settlement  $S$  were obtained when  $H$  was assumed to be 34.0 ft.

The expressions then become

$$S_{es} = 7.25 \log \left( 1 + \frac{\Delta p}{1.248} \right) \quad (10)$$

$$S_{ms} = 4.25 \log \left( 1 + \frac{\Delta p}{1.354} \right) \quad (11)$$

Here  $\Delta p$  is the stress, in tons per sq ft, under the point for which settlement is being computed, at mid-height of the 34-ft layer, or in other words at El. -20.0. The settlement  $S$  is in feet.

### 37. Approximate Linear Relation

The dash curves in Fig. 60 represent Eqs. 10 and 11 graphically. The figure also indicates the computed settlements for all points on the build-

ings previously discussed, plotted as a function of the stress  $\Delta p$  at El. -20. Seventy-seven percent of all such points fall within the limiting values of the approximate method. It is noticed in Fig. 60 that the lower limiting curve actually defines the minimum settlements for all the buildings discussed in the previous chapters. All the points above the upper limiting curve are concentrated between  $\Delta p = 0$  to 0.35 tons per sq ft, whereas all the points with higher stress at El. -20.0 are in the region between the two limiting curves. An average straight line through all these points can be assumed as a fair approximation to the relation between the settlement and the stress at El. -20.0. The equation of this line is

$$S \text{ (ft)} = 0.20 + 1.34 \Delta p \text{ (tons per sq ft)} \quad (12)$$

where  $\Delta p$  is the net stress at El. -20.0 due to the building and excavation loads. This straight line gives the computed settlement within about 0.2 ft

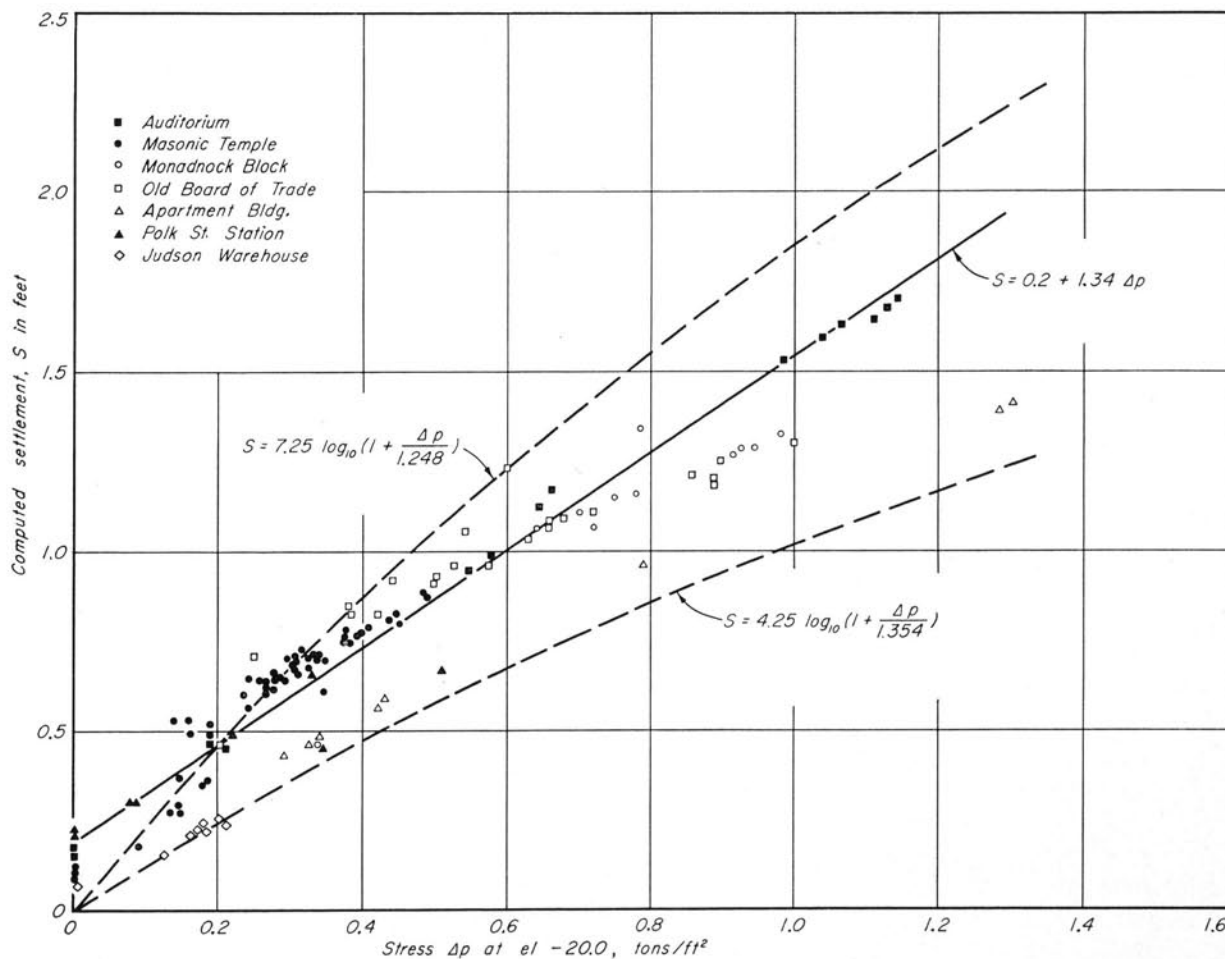


Fig. 60. Comparison of Primary Settlements Computed by Conventional Methods and Simplified Theory



of the value found by the general procedure. The computed differential settlements for a particular building are given with much greater accuracy.

The straight line, Eq. 12, represents the computed settlements with satisfactory accuracy. It is, consequently, pertinent to investigate whether it also agrees in a satisfactory manner with the actual settlements. Figure 61 provides this information. In this figure, each plotted point represents the observed settlement of some point on one of the seven buildings, and the corresponding stress  $\Delta p$  at El. -20.0.

Examination of Fig. 61 leads to the conclusion that the agreement between the observed settlements and those predicted in accordance with the approximate linear relationship is almost as satisfactory as that between the settlements computed by means of the general and the approximate procedures. The average settlement of a building is given with an error usually less than 0.3 ft. If the building is fairly flexible, the differential settlements agree with the approximate computed ones with even greater accuracy. If the building is very stiff, the real differential settlements are inevitably much less than those indicated by Eq. 12. For example, the points representing the extremely rigid Apartment Building show considerable scattering from the straight line. Even so, the average settlement of the building is in error by only 0.12 ft.

### 38. Summary and Discussion

The principal purpose of a settlement forecast is to provide the designer of a structure with a means for determining whether or not the foundation that he contemplates will be adequate for its purpose. The forecast serves as a guide to his judgment; it cannot replace it. As a consequence, extreme accuracy is not necessary. The practical value of the computed settlements depends largely upon the designer's ability to interpret the results and to infer from them the probable behavior of his structure.

For example, the differential settlement of a building depends not only upon the characteristics of the subsoil but also, to a very great extent, on the rigidity of the building. Relatively few modern buildings approach a condition of complete flexibility, yet every practicable method of settlement computation contains the assumption that there is no external restriction to the manner of deformation of the ground surface. Attempts to evaluate and consider the effect of building stiffness quanti-

tatively have led to complexity far beyond any benefits, and to results of doubtful accuracy on account of the assumptions involved. However, if the designer knows the order of magnitude of the average settlement, and the order of magnitude and general pattern of the differential settlement of a flexible structure otherwise identical to his, he can intelligently judge the effects, both on the structure and on the settlement pattern, of the rigidity of his proposed structure.

Therefore, a settlement forecast usually serves its purpose if it reliably indicates the order of magnitude of the average settlement, and if it gives a reliable conception of the differential settlement of a flexible structure otherwise identical to the proposed one.

The reliability of the settlement computations made in the manner described in this bulletin for Chicago buildings can be judged most readily by means of Fig. 61, which shows the actual settlements of some 113 reference points on 7 Chicago buildings as well as the straight line that represents the settlements computed on the basis of Eq. 12. The straight line itself is a rather crude modification of the results of the general theory, because the settlement is assumed to depend exclusively on the stress  $\Delta p$ , caused by the building, at El. -20.0. Yet, because Fig. 61 summarizes all the preceding data, it is a satisfactory starting point for discussion.

Very obvious is the fact that the one rigid building included in the study, the reinforced concrete Apartment Building, did not settle according to the linear relation, and the individual reference points show a broad scattering. This, of course, is to be expected, because the differential settlement of this building was almost entirely a function of the properties of the structure rather than of the soil. Nevertheless, the point representing the average settlement of this building is close to the straight line. Thus, even for this structure the average settlement is reliably given.

By contrast, the Auditorium was in reality an extremely flexible building. The points in Fig. 61 corresponding to this structure tend to fall into three groups: those representing the lightly loaded interior columns (see Fig. 10), grouped about the  $\Delta p = 0$  axis; those representing the moderately loaded rear wall, grouped about  $\Delta p = 0.6$  ton per sq ft; and those representing the heavily loaded tower, grouped about  $\Delta p = 1.1$  tons per sq ft. Although the tower within itself was rigid, these three elements of the structure were free to settle

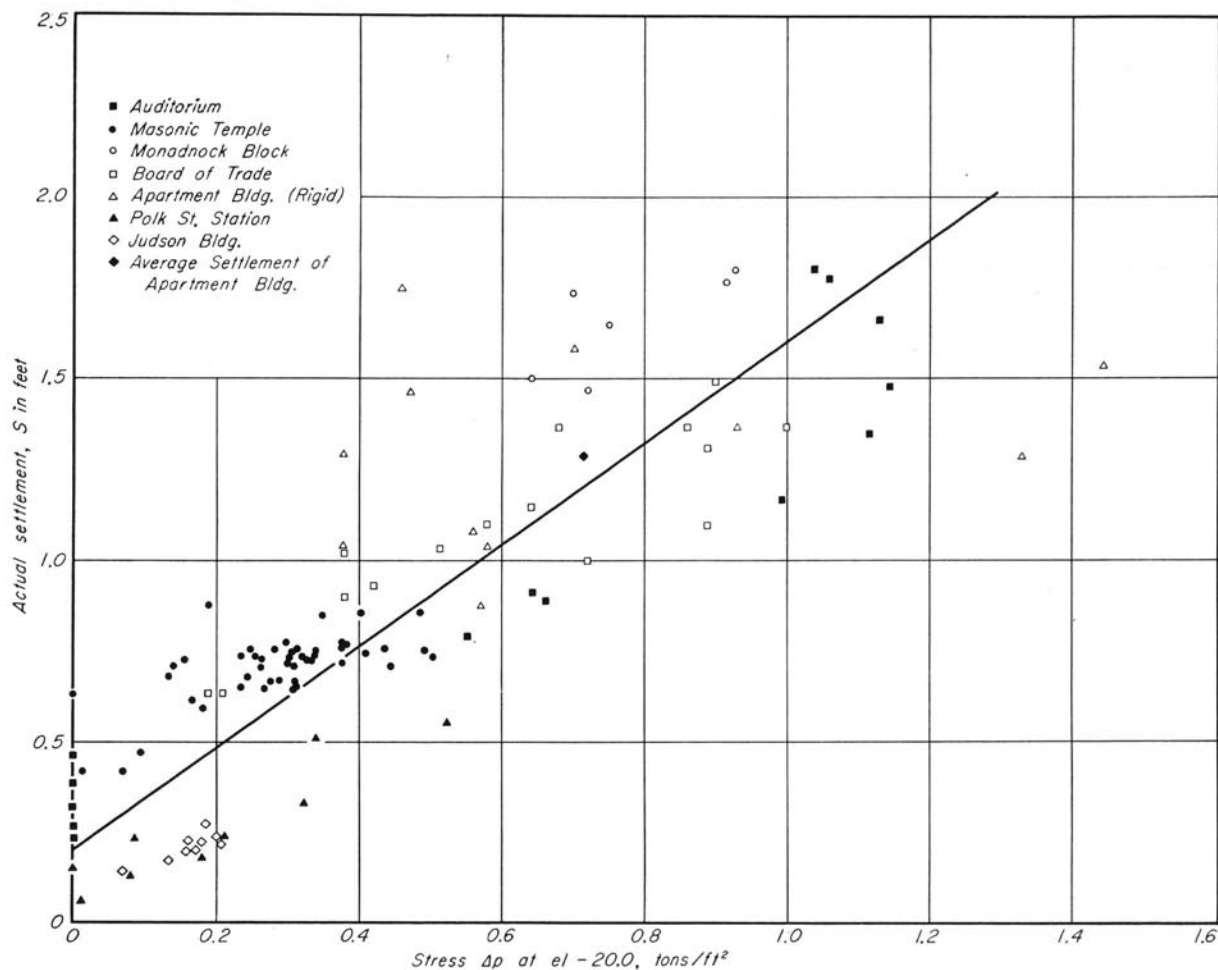


Fig. 61. Relation of Observed Settlement to Change in Stress at El. -20

almost independently. Throughout the entire range of stress  $\Delta p$ , which embraces beneath this structure the least to the greatest unit pressures found at this elevation under all the buildings studied, the agreement between the computed and the actual settlements is very close. Likewise, points representing the settlement of the Old Board of Trade, another unusually flexible building, are in good agreement with the computed values.

Points representing the Monadnock Block, which may be classified as a moderately flexible building because of inevitable plastic flow in the masonry over a period of years, are located on the average farther from the straight line than those corresponding to any other building. This may be due in part to softer than average soil conditions, to uncertainties regarding the initial elevation of the building, or to other unknown causes. However, the points lie on a line roughly parallel to the linear relation and, consequently, the differential settlements are quite accurately given.

The Polk Street Station, the Masonic Temple, and the Judson Warehouse all stressed the soil to a much shallower depth, and to a much smaller value in comparison with the unconfined compressive strength of the soil, than did the other buildings. This is obvious from a study of the stress bulbs, Figs. 10, 20, 28, 42, 50, 55 and 59. Of these buildings, the Masonic Temple created the greatest stresses, but their variation was relatively small at a depth as great as El. -20. Therefore, the differential settlements between most of the points were less than those indicated by the semi-empirical relation. Yet, the computed maximum differential settlement of 0.7 ft compared favorably with the actual differential settlement of 0.6 ft. The differential settlements of the Polk Street Station and the Judson Warehouse, both relatively flexible structures, were quite accurately predicted, but the soil was stressed to such a shallow depth that the average computed settlements were about 0.2 ft too great.

The preceding discussion suggests that the general order of accuracy of the total and differential computed settlements is satisfactory for practical purposes. It is pertinent to try to evaluate the ratio of computed to observed settlement on a quantitative basis. This can best be done by utilizing the results of the detailed settlement computations rather than those of the approximate procedure. If consideration is given only to the five buildings for which enough time-settlement data are available to permit a fairly reliable estimate of the value of 100 percent-consolidation settlement, the ratios of computed to observed average settlements are approximately as shown in Table 14. According to this table the observed settlements were on the average about 15 percent greater than the computed ones. The excess may represent the additional, uncomputed settlement due to the lateral deformation of the clay. If so, this factor is not of great practical importance in connection with Chicago conditions, and it may be concluded that the use of the confined compression (standard consolidation) test is entirely justified even if the compressible soil beneath the footings is not confined between two relatively inextensible layers.

Before a final conclusion is reached on this point, however, it is desirable to investigate the possible influence on the settlement of some of the other assumptions that enter into the calculations. These assumptions directly influence either the computed stresses or the compressibility of the clay. It may be shown that a variation of 10 percent in the computed stresses causes a variation of about 6 percent in the computed settlement. If the loads have been properly evaluated, it is unlikely that the computed stresses should be in error appreciably more than 10 percent. On the other hand, a variation of 1 percent in the average water content of the consolidating clay strata leads to a variation of about 5 percent in  $C_c$  and a variation of about 6 percent in the computed settlement. On the basis of soil variation studies made in this region (Peck 1940), it seems quite possible that the average water content obtained from one boring may differ by 2 or 3 percent from that obtained from another only a few feet away. Hence, the computed settlements could easily be in error by 15 percent because of this factor. However, the average water content should not be in error by over about 4 percent even in the most variable parts of the deposit. This

would correspond to a maximum error of about 25 percent in the computed settlement.

Some additional insight into the possible variation in compressibility of the soil may be obtained from a study of the settlement curves of the Old Board of Trade. The trading-hall portion of this building was, as far as can be determined, symmetrical and symmetrically loaded. Yet points III and III-A had settled respectively 0.516 and 0.380 ft by 1895. This difference of 27 percent is most likely due to variations in soil conditions; all other known conditions were identical.

Therefore, it must be concluded that errors due to improper evaluation of the compressibility of the soil may easily be of the same order of magnitude as the observed difference between computed and

Table 14  
Ratio of Computed to Observed Settlement  
for Principal Structures

Building	Ratio (%)
Auditorium	95
Masonic Temple	82
Monadnock Block	72
Old Board of Trade	92
Apartment Building	83
Average	85

real settlements. Yet, if the errors are due to random variations in soil properties, they should be sometimes positive and sometimes negative. The fact that the computed settlements are consistently less than the observed ones (except in the case of the Polk Street Station for which the data concerning soil conditions are uncertain) would seem to indicate that at least some of the difference may be due to the effect of incomplete lateral confinement.

In any event, the discrepancy between observed and computed settlements is far less important than the secondary time effect, which is not included in the computations. A study of this effect is not within the scope of this bulletin, but its importance was recognized at an early stage of the investigation. It was also observed that the resemblance between the time-settlement curves for the buildings and those obtained by means of laboratory tests is sometimes very remote. Both these subjects deserve further study.

In conclusion, it may be said that the primary settlement of buildings founded above the soft clay deposit in Chicago can be predicted with sufficient accuracy for practical purposes by means of the general procedure customarily used for deep-seated, laterally confined deposits of clay.

## X. RELATION BETWEEN SETTLEMENT AND BEARING CAPACITY

### 39. Theoretical Relation

In the preceding chapters the data concerning seven structures in Chicago have been used as a basis for studying the reliability of what may be termed the conventional methods for computing the settlement of structures established above deposits of clay. The methods make possible an estimate of not only the maximum settlement but also the differential settlements to be expected.

The data from some of the structures may also be used, however, to investigate the reliability of much simpler theoretical procedures for estimating the ultimate or final average settlement of a structure having a simple geometrical ground plan and exerting fairly uniform pressure on the subsoil. The theory (Skempton 1951), based essentially on considerations of the behavior of an elastic solid, leads to the following expression

$$\frac{S}{B} = \frac{5}{K_v/c} \frac{q}{q_d} \quad (13)$$

where

$S$  = average ultimate settlement of structure

$B$  = width of structure

$K_v$  = modulus of rigidity of subsoil, the reciprocal of  $m_v$ , the modulus of compressibility. The latter quantity is determined from an  $e$ - $p$  curve in a standard consolidation test by means of the expression

$$m_v = \frac{1}{1 + e_0} \frac{e_1 - e_0}{p_1 - p_0} \quad (14)$$

$c$  = cohesion of the clay, taken as half the unconfined compressive strength

$q$  = net soil pressure at the base of the structure

$q_d$  = ultimate bearing capacity of the clay beneath the structure

Experience has suggested that the ratio  $K_v/c$  is approximately constant for the clays in a given geological formation or group of formations of similar origin. Moreover, the ratio  $q_d/q$  is the factor of safety against a bearing-capacity failure. Hence,

if  $K_v/c$  is actually constant for a locality such as downtown Chicago, Eq. 13 reduces to

$$\frac{S}{B} = \text{const.} \frac{1}{F.S.} \quad (15)$$

That is, points representing values of settlement per unit width of the structures, when plotted as a function of the reciprocal of the factor of safety, should define a straight line through the origin. Moreover, the slope of the line should be characteristic of the locality.

### 40. Evaluation for Conditions in Chicago

The information required for plotting points characteristic of structures in Chicago consists of the following items for each building:

1. The average final settlement; that is, the settlement up to the time when movement has virtually stopped
2. The width of the loaded area
3. The net soil pressure
4. The ultimate bearing capacity of the subsoil.

The data for evaluating the first three items are contained in the records that have been presented previously for the various buildings. The net ultimate bearing capacity for a structure on a clay subsoil is given with considerable accuracy by the expression

$$q_d = c N_c \quad (16)$$

where  $c$  is the cohesion, and  $N_c$  is the bearing capacity factor. The factor  $N_c$  (Skempton 1951) is given by

$$N_c = 5 \left( 1 + 0.2 \frac{B}{L} \right) \left( 1 + 0.2 \frac{D_f}{B} \right) \quad (17)$$

where  $L$  is the length of the foundation and  $D_f$  is the depth of the base of the foundation below the surrounding ground level. These quantities are also known for the various structures.

The value of cohesion to be substituted in Eq. 16 is the average cohesion from the base of the foundation to the bottom of the compressible clay. It may



be determined by computing the average unconfined compressive strength and dividing the result by two.

Values of the unconfined compressive strength characteristic of each layer beneath each of the structures considered in this study are shown in a set of diagrams (Figs. 10, 20, 28, 42, 50, 55 and 59). The strength of the stiff upper crust is taken arbitrarily as 1.0 ton per sq ft. The weighted average value of  $q_u$  for each structure, as determined from the diagrams, is the value corresponding to tests on 2-in. Shelby tube samples. It must be multiplied by 1.35 to obtain the corresponding value for the undisturbed soil (Peck and Reed 1954). One half the corrected average value represents the cohesion to be used for computation of the ultimate bearing capacity.

Not all the structures discussed in the preceding chapters are suitable for consideration in the present study. The factors leading to inclusion or exclusion of the individual buildings are as follows:

**Auditorium:** Although this structure has a very non-uniform distribution of loads over the area as a whole, the tower has a fairly uniformly loaded rectangular base and is included.

**Masonic Temple:** This structure is rectangular in shape and fairly uniformly loaded. It is suitable for inclusion.

**Monadnock Block:** This structure likewise is uniformly loaded on a rectangular base, and is included.

**Old Board of Trade:** The widely varying intensity of loading over the area of the building makes this structure unsuitable.

**Apartment Building:** The T-shaped plan of this structure and the eccentric loading introduce several uncertainties into the analysis and make the structure unsuited to the study.

**Polk St. Station:** The structure as a whole is loaded with varying intensities and has an irregular plan. However, the tower footing is a square and is included.

**Judson Warehouse:** The presence of the fill beneath the floor on the west half of the structure produces an eccentric loading and renders the structure unsuitable for the study.

The pertinent data for the four structures chosen for the study are given in Table 15. The values of  $S/B$  and  $1/F.S.$  are plotted in Fig. 62. Corresponding values of the factor of safety are shown on the right side of the figure.

The four points define a straight line through the origin in agreement with the theory expressed by Eq. 15 and in agreement with the assumption that the value of  $K_v/c$  for the subsoil of the downtown section of Chicago is essentially constant. The validity of the relationship, moreover, has been demonstrated over an exceptionally wide range in values of the factor of safety against a bearing-capacity failure.

Table 15  
Data for Computation of Factor of Safety and Settlement  
per Unit Width of Principal Buildings

	Auditorium Tower	Monadnock Block	Polk Street Sta. Tower	Masonic Temple
Breadth of foundation $B$	67	91	34	142
Length of foundation $L$	100	412	34	197
Depth of foundation $D_f$	18	12	14	14
Gross pressure	4100	3200	3300	2020
Excavation load	1580	1260	1350	1350
Net pressure	2520	1940	1950	670
$D_f/B$	0.268	0.132	0.41	0.10
$N_c$	5.7	5.3	6.1	5.8
Corrected cohesion $c$	732	815	840	752
Ultimate bearing capacity $q_u$	4180	4320	5150	4370
Factor of safety	1.67	2.24	2.50	6.7
Final settlement	2.24	1.90	0.67	1.0
$S/B$ (%)	3.33	2.08	2.00	0.70

$c = 1.35 \times \frac{1}{2}$  (unconfined compressive strength of 2-in. tube samples)

All lengths in feet; all pressures in lb per ft<sup>2</sup>

#### 41. Evaluation of Ratio of Rigidity to Strength

The diagram, Fig. 62, also contains lines representing various values of  $K_v/c$  as determined by Eq. 13. According to the position of the plotted points, the value appropriate to the buildings in Chicago is  $K_v/c = 90$ . It is possible to compute the values of this ratio independently for various points beneath each building on the basis of the results of soil tests.

The quantity  $K_v$  is the reciprocal of the coefficient of volume compressibility  $m_v$  (Eq. 14). For a normally loaded clay, the coefficient  $m_v$  may be expressed (Terzaghi 1943) as

$$m_v = \frac{C_c}{1 + e_0} \frac{1}{\Delta p} \log_{10} \left( 1 + \frac{\Delta p}{p_0} \right) \quad (18)$$

Every quantity on the right-hand side of this equation has been evaluated at mid-height of each layer beneath each point for which settlements have been computed in connection with the structures considered in Chapters II to VIII inclusive. For these same layers the values of unconfined compressive strength and, hence, cohesion are known. Thus, the data are available for computation of the ratio  $K_v/c$ . The computations are shown in Table 16 for one point beneath each of the four buildings suitable for the study. The points chosen were those considered most representative of the settlement



of the structure; namely, point *d* on the Auditorium Tower, columns 33/34/78 of the Masonic Temple, point *j* of the Monadnock Block, and point *e* of the tower of the Polk Street Station.

The values of the ratio  $K_v/c$  are found to be quite consistent for all the structures at all depths. The weighted average values are as follows:

Auditorium Tower	90
Monadnock Block	77
Masonic Temple	70
Polk Street Station Tower	69
Average	77

These values are inscribed in Fig. 62. They indicate that, in general, the ratio  $K_v/c$  as determined by computation from the results of soil tests is of the same order of magnitude but somewhat smaller than that based on Eq. 13 and the records of settlement and loading for the various structures. Similar results for other localities have been reported by Skempton.

## 42. Conclusion

The studies described in this chapter demonstrate that Eq. 13, which represents a theoretical relationship between bearing capacity, soil pressure and settlement, is valid for the conditions encountered in Chicago. If the value 90 is assigned to the

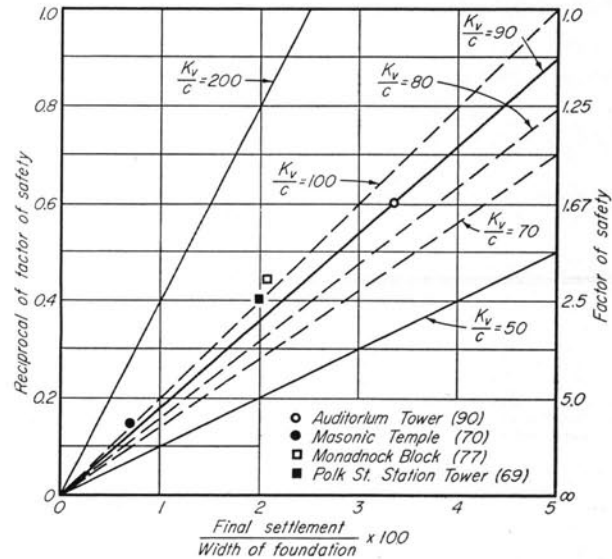


Fig. 62. Relationship Between Settlement per Unit Width of Foundation and Factor of Safety against Bearing-Capacity Failure

ratio  $K_v/c$ , the resulting expression

$$\frac{S}{B} = \frac{1}{18} \frac{q}{q_d} \quad (19)$$

may be regarded as a highly reliable semi-empirical expression for estimating the final average settlement of a regularly shaped, uniformly loaded structure established above the clay deposits in downtown Chicago.

Table 16  
Data for Computation of Ratio  $K_v/c$  for Principal Buildings

Building	Point	Layer	$e_0$	$\frac{C_c}{1+e_0}$	$p_0$	$\Delta p$	$m_v$	$q_u$	$c$	$K_v/c$
Auditorium	<i>d</i>	B	1.008	0.203	1.054	1.150	0.057	0.32	0.21	83
		C	1.260	0.260	1.304	1.147	0.062	0.26	0.17	95
		D	0.672	0.133	1.554	1.088	0.028	0.62	0.42	85
		E	0.672	0.133	1.956	0.986	0.024	0.62	0.42	99
Masonic Temple	33/ 34/ 78	B	0.812	0.162	0.813	0.754	0.061	0.44	0.30	55
		C	0.812	0.162	0.944	0.708	0.056	0.44	0.30	60
		D	0.812	0.162	1.127	0.640	0.049	0.44	0.30	68
		E	0.812	0.162	1.281	0.578	0.045	0.44	0.30	73
		F	0.672	0.133	1.514	0.382	0.034	0.62	0.42	70
		G	0.672	0.133	1.814	0.257	0.029	0.62	0.42	82
Monadnock Block	<i>j</i>	B	0.756	0.150	0.869	1.104	0.048	0.50	0.34	61
		C	0.756	0.150	1.061	0.983	0.043	0.50	0.34	68
		D	0.644	0.127	1.289	0.948	0.032	0.68	0.46	68
		E	0.644	0.127	1.554	0.885	0.028	0.68	0.46	78
		F	0.644	0.127	1.817	0.805	0.025	0.68	0.46	87
		G	0.644	0.127	2.081	0.747	0.023	0.68	0.46	94
Polk Street Station	<i>e</i>	B	1.008	0.203	0.822	0.920	0.072	0.33	0.22	63
		C	1.008	0.203	0.945	0.820	0.067	0.33	0.22	68
		D	0.644	0.127	1.161	0.705	0.037	0.68	0.46	59
		E	0.644	0.127	1.486	0.531	0.032	0.68	0.46	68
		F	0.644	0.127	1.827	0.347	0.027	0.68	0.46	80

$e_0$  = initial void ratio

$C_c/(1+e_0)$  = compression ratio, Fig. 2

$p_0$  = initial overburden pressure, ton per ft<sup>2</sup>

$\Delta p$  = net increase in pressure, ton per ft<sup>2</sup>

$m_v$  = coefficient of volume compressibility =  $1/K_v$ , ft<sup>2</sup> per ton

$q_u$  = unconfined compressive strength of 2-in. tube samples, ton per ft<sup>2</sup>

$c$  = corrected cohesion =  $1.35 \times q_u/2$ , ton per ft<sup>2</sup>

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